Vertical Interior Post and Horizontal Rail Design

Given:

Post Height:	$H_{post} := 42\ in$	Plans
Post and Rail Yield Strength:	$F_y := 50\ ksi$	ASTM F1043
Post and Rail Modulus of Elasticity:	$E_s := 29000\ ksi$	
Post and Rail Ultimate Strength:	$F_u := 58\ ksi$	

Properties for ASTM F1043 IC 2-7/8" Pipe for Interior Posts

Post OD:	$OD_{post} := 2.875\ in$
Post Thickness:	$t_{post} := 0.160\ in$

Properties for ASTM F1043 IC 1-5/8" Pipe for Rails

Rail OD:	$OD_{rail} := 1.66\ in$
Rail Thickness:	$t_{rail} := 0.111\ in$

Design Point Live Load	$P_{LL} := 200\ lbf$	AASHTO 13.8.2
Design Uniform Live Load	$w_{LL} := 50\ plf$	AASHTO 13.8.2
Post spacing:	$L_{spc} := 6\ ft$	Plans
Weight of chain link fence:	$f_{clf} := 0.48\ psf$	
Design wind load from chain link fence:	$f_{wind} := 15\ psf$	AASHTO 13.8.2

Load Factors (AASHTO Tbl. 3.4.1-1):

PL Load Factor:	$\gamma_{PL} := 1.75$
DC Load Factor:	$\gamma_{DL} := 1.25$
WS Load Factor:	$\gamma_{WS} := 1.00$

Resistance Factors:

Steel Flexure (AASHTO 6.5.4.2):	$\phi_f := 1.00$
Steel Shear (AASHTO 6.5.4.2):	$\phi_v := 1.00$
Tension, Yielding in Gross Section:	$\phi_y := 0.95$
Bending (AISC F1):	$\phi_b := 0.90$
Shear (AISC G1):	$\phi_{v\_AISC} := 0.90$
Bearing (AISC DG#1):	$\phi_{brg} := 0.60$
Fillet Weld (AISC Tbl. J2.5):	$\phi_{fw} := 0.75$
Bolts (AISC J3.6 & J3.7):	$\phi_{ab} := 0.75$
Adhesive Anchor Bolts (ACI 17.3.3, Condition B, Category 1):	$\phi_{adh} := 0.65$

Steel weight density: $\gamma_{steel} := 490\ pcf$

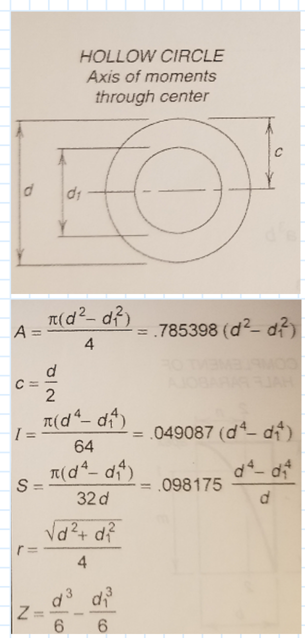
ASTM F1043 Group IC Electric Resistant Welded 50,000 psi yield steel pipe

Trade Reference	Decimal O.D. Equivalent		Pipe wall Thickness		Weight		Section Modulus			Min. Yield Strength			Max Bending Moment	Calculated Load (lbs)		
									x			=		10' Free	Cantilever	
O.D.	inches	(mm)	inches	(mm)	lb./ft.	(kg/m)	inches³	(mm³)	x	psi	(Mpa)	=	Lb.In.	Supported	4'	6'
1 5/8"	1.660	42.16	0.111	2.82	1.84	2.74	0.1962	4.98	x	50000	345	=	9810	327	204	136
1 7/8"	1.900	48.26	0.120	3.05	2.28	3.39	0.2810	7.14	x	50000	345	=	14050	468	293	195
2 3/8"	2.375	60.33	0.130	3.30	3.12	4.64	0.4881	12.40	x	50000	345	=	24405	814	508	339
2 7/8"	2.875	73.03	0.160	4.06	4.64	6.91	0.8778	22.30	x	50000	345	=	43890	1463	914	610
3 1/2"	3.500	88.90	0.160	4.06	5.71	8.50	1.3408	34.06	x	50000	345	=	67042	2235	1397	931
4"	4.000	101.60	0.160	4.06	6.57	9.78	1.7820	45.26	x	50000	345	=	89098	2970	1856	1237
4 1/2"	4.500	114.30	0.160	4.14	7.42	11.04	2.2859	57.99	X	50000	345	=	114295	3810	5486	1587

Output:

Post Section Properties:

Post inside diameter:	$ID_{post} := OD_{post} - 2 \cdot t_{post}$	$ID_{post} = 2.555\ in$
Post Area:	$A_{post} := 0.785398 \cdot \left(OD_{post}^2 - ID_{post}^2\right)$	$A_{post} = 1.365\ in^2$
Post Unit Weight:	$w_{post} := \gamma_{steel} \cdot A_{post}$	$w_{post} = 4.644\ plf$
Post centroid:	$c_{post} := 0.5 \cdot OD_{post}$	$c_{post} = 1.438\ in$
Post Moment of Inertial:	$I_{post} := 0.049087 \cdot \left(OD_{post}^4 - ID_{post}^4\right)$	$I_{post} = 1.262\ in^4$
Post Section Modulus:	$S_{post} := \dfrac{I_{post}}{c_{post}}$	$S_{post} = 0.878\ in^3$
Post Plastic Section Modulus:	$Z_{post} := \dfrac{OD_{post}^3 - ID_{post}^3}{6}$	$Z_{post} = 1.181\ in^3$



Rail Section Properties:

Rail inside diameter:	$ID_{rail} := OD_{rail} - 2 \cdot t_{rail}$	$ID_{rail} = 1.438\ in$
Rail Area:	$A_{rail} := 0.785398 \cdot \left(OD_{rail}^2 - ID_{rail}^2\right)$	$A_{rail} = 0.54\ in^2$
Rail Unit Weight:	$w_{rail} := \gamma_{steel} \cdot A_{rail}$	$w_{rail} = 1.838\ plf$
Rail centroid:	$c_{rail} := 0.5 \cdot OD_{rail}$	$c_{rail} = 0.83\ in$
Rail Moment of Inertial:	$I_{rail} := 0.049087 \cdot \left(OD_{rail}^4 - ID_{rail}^4\right)$	$I_{rail} = 0.163\ in^4$
Rail Section Modulus:	$S_{rail} := \dfrac{I_{rail}}{c_{rail}}$	$S_{rail} = 0.196\ in^3$
Rail Plastic Section Modulus:	$Z_{rail} := \dfrac{OD_{rail}^3 - ID_{rail}^3}{6}$	$Z_{rail} = 0.267\ in^3$

Post concentrated live load applied at top rail: $P_{post\_LL} := P_{LL} + w_{LL} \cdot L_{spc} = 0.5\ kip$ $P_{post\_LL} = 0.5\ kip$ *AASHTO Eqn. 13.8.2-1*

Post moment loading from live load: $M_{post\_LL} := P_{post\_LL} \cdot H_{post} = 21000\ lbf \cdot in$ $M_{post\_LL} = 21000\ lbf \cdot in$ *Post treated as cantilevered beam*

Post shear from live load: $V_{post\_LL} := P_{post\_LL}$ $V_{post\_LL} = 0.5\ kip$

Rail moment from live load applied: $M_{rail\_LL} := \sqrt{2} \cdot \frac{w_{LL} \cdot L_{spc}^{2}}{8} + \frac{P_{LL} \cdot L_{spc}}{4}$ $M_{rail\_LL} = 7418.377\ lbf \cdot in$ *Rail treated as simply supported beam with vertical and horizontal live loads combined into resultant direction.*

Rail moment from dead load: $M_{rail\_DL} := \frac{w_{rail} \cdot L_{spc}^{2}}{8} + \frac{f_{clf} \cdot \frac{H_{post}}{2} \cdot L_{spc}^{2}}{8}$ $M_{rail\_DL} = 144.615\ lbf \cdot in$

Rail shear from live load: $V_{rail\_LL} := \sqrt{2} \cdot \frac{w_{LL} \cdot L_{spc}}{2} + \frac{P_{LL}}{2}$ $V_{rail\_LL} = 0.312\ kip$

Rail shear from dead load: $V_{rail\_DL} := \frac{w_{rail} \cdot L_{spc}}{2} + \frac{f_{clf} \cdot \frac{H_{post}}{2} \cdot L_{spc}}{2}$ $V_{rail\_DL} = 0.008\ kip$

Factored Shear Load on Post: $V_{post\_u} := \gamma_{PL} \cdot V_{post\_LL}$ $V_{post\_u} = 0.875\ kip$

Factored Moment Load on Post: $M_{post\_u} := \gamma_{PL} \cdot M_{post\_LL}$ $M_{post\_u} = 36750\ lbf \cdot in$

AASHTO load factors used instead of ASCE load factors found in AISC and ACI. This is acceptable as it is more conservative.

Factored Shear Load on Rail: $V_{rail\_u} := \gamma_{PL} \cdot V_{rail\_LL} + \gamma_{DL} \cdot V_{rail\_DL}$ $V_{rail\_u} = 0.556\ kip$

Factored Moment Load on Rail: $M_{rail\_u} := \gamma_{PL} \cdot M_{rail\_LL} + \gamma_{DL} \cdot M_{rail\_DL}$ $M_{rail\_u} = 13162.928\ lbf \cdot in$

Vertical dead load was combined directly with live load resultant since it was so small compared to the live load.

Post Analysis:

Following AASHTO 6.12.1.2.3c for Shear Design:

Gross Area: $A_g := A_{post}$ $A_g = 1.365\ in^2$

Distance from Max to 0 Shear: $L_v := H_{post}$ $L_v = 42\ in$

Critical Strength for Shear:

$$F_{cr} := min\left(0.58 \cdot F_y , \max\left(\frac{1.6 \cdot E_s}{\sqrt{\frac{L_v}{OD_{post}}}\left(\frac{OD_{post}}{t_{post}}\right)^{\frac{5}{4}}}, \frac{0.78 \cdot E_s}{\left(\frac{OD_{post}}{t_{post}}\right)^{\frac{3}{2}}}\right)\right)$$

$F_{cr} = 29\ ksi$ *AASHTO Eqns. 6.12.1.2.3c-2 & 6.12.1.2.3c-3*

Factored nominal shear resistance: $\phi V_n := \phi_v \cdot 0.5\ F_{cr} \cdot A_g$ $\phi V_n = 19.788\ kip$ *AASHTO Eqn. 6.12.1.2.3c-1*

Post Shear Check: $\frac{\phi V_n}{V_{post\_u}} = 22.615$

$Post\_Shear\_Check :=$ if $\frac{\phi V_n}{V_{post\_u}} \geq 1.0$
  "Post shear strength is satisfactory."
else
  "Post is not satisfactory."

$Post\_Shear\_Check =$ "Post shear strength is satisfactory."

Following AASHTO 6.12.2.2.3 for Flexure Design:

Check of Noncompact Section:

$Check\_Compact :=$ if $\frac{OD_{post}}{t_{post}} \leq \frac{0.07 \cdot E_s}{F_y}$
  "Section is compact. Local buckling does not apply."
else
  "Section is not compact. Check wall slenderness."

Per AASHTO 6.12.2.2.3, as long D/t does not exceed 0.07E/Fy, plastic modulus and equation 6.12.2.2.3-1 may be used.

$Check\_Compact =$ "Section is compact. Local buckling does not apply."

Factored Nominal Moment Resistance: $\phi M_n := \phi_f \cdot F_y \cdot Z_{post}$ $\phi M_n = 59.038\ kip \cdot in$ *AASHTO Eqn. 6.12.2.2.3-1*

Post Flexural Check:

$$\frac{\phi M_n}{M_{post\_u}} = 1.606$$

$$Post\_Flex\_Check := \text{if } \frac{\phi M_n}{M_{post\_u}} \geq 1.0$$
‖ "Post flexural strength is satisfactory."
else
‖ "Post is not satisfactory."

$Post\_Flex\_Check =$ "Post flexural strength is satisfactory."

Rail Analysis:

Following AASHTO 6.12.1.2.3c for Shear Design:

Gross Area: $A_g := A_{rail}$ $A_g = 0.54\ in^2$

Distance from Max to 0 Shear: $L_v := \frac{L_{spc}}{2}$ $L_v = 36\ in$

Critical Strength for Shear:

$$F_{cr} := min\left(0.58 \cdot F_y, \max\left(\frac{1.6 \cdot E_s}{\sqrt{\frac{L_v}{OD_{rail}}}\left(\frac{OD_{rail}}{t_{rail}}\right)^{\frac{5}{4}}}, \frac{0.78 \cdot E_s}{\left(\frac{OD_{rail}}{t_{rail}}\right)^{\frac{3}{2}}}\right)\right)$$

$F_{cr} = 29\ ksi$ *AASHTO Eqns. 6.12.1.2.3c-2 & 6.12.1.2.3c-3*

Factored Nominal Shear Resistance: $\phi V_n := \phi_v \cdot 0.5\ F_{cr} \cdot A_g$ $\phi V_n = 7.832\ kip$ *AASHTO Eqn. 6.12.1.2.3c-1*

Rail Shear Check:

$$\frac{\phi V_n}{V_{rail\_u}} = 14.08$$

$$Rail\_Shear\_Check := \text{if } \frac{\phi V_n}{V_{rail\_u}} \geq 1.0$$
‖ "Rail shear strength is satisfactory."
else
‖ "Rail is not satisfactory."

$Rail\_Shear\_Check =$ "Rail shear strength is satisfactory."

Following AASHTO 6.12.2.2.3 for Flexure Design:

Check of Noncompact Section:

$$Check\_Compact := \text{if } \frac{OD_{rail}}{t_{rail}} \leq \frac{0.07 \cdot E_s}{F_y}$$
‖ "Section is compact. Local buckling does not apply."
else
‖ "Section is not compact. Check wall slenderness."

Per AASHTO 6.12.2.2.3, as long D/t does not exceed 0.07E/Fy, plastic modulus and equation 6.12.2.2.3-1 may be used.

$Check\_Compact =$ "Section is compact. Local buckling does not apply."

Factored Nominal Moment Resistance: $\phi M_n := \phi_f \cdot F_y \cdot Z_{rail}$ $\phi M_n = 13.339\ kip \cdot in$ *AASHTO Eqn. 6.12.2.2.3-1*

Post Flexural Check:

$$\frac{\phi M_n}{M_{rail\_u}} = 1.013$$

$$Rail\_Flex\_Check := \text{if } \frac{\phi M_n}{M_{rail\_u}} \geq 1.0$$
‖ "Rail flexural strength is satisfactory."
else
‖ "Rail is not satisfactory."

$Rail\_Flex\_Check =$ "Rail flexural strength is satisfactory."

Confirming that Wind Loading Doesn't Control:

Per last paragraph of AASHTO 13.8.2, the wind load on the chain link fence is not applied simultaneously with the live load.

Uniform wind load on post: $w_{post\_wind} := f_{wind} \cdot L_{spc}$ $w_{post\_wind} = 90\ plf$

Design moment from wind on post: $M_{post\_wind\_u} := \gamma_{WS} \cdot \frac{w_{post\_wind} \cdot H_{post}^{2}}{2}$ $M_{post\_wind\_u} = 6615\ lbf \cdot in$

$M_{post\_u} = 36750\ lbf \cdot in$ <- LL controls

Design shear from wind on post: $V_{post\_wind\_u} := \gamma_{WS} \cdot w_{post\_wind} \cdot H_{post}$ $V_{post\_wind\_u} = 0.315\ kip$

$V_{post\_u} = 0.875\ kip$ <- LL controls

Uniform wind on rail: $w_{rail\_wind} := f_{wind} \cdot \frac{H_{post}}{2}$ $w_{rail\_wind} = 26.25\ plf$

Design moment from wind on rail: $M_{rail\_wind\_u} := \gamma_{WS} \cdot \frac{w_{rail\_wind} \cdot L_{spc}^{2}}{8}$ $M_{rail\_wind\_u} = 1417.5\ lbf \cdot in$

$M_{rail\_u} = 13162.928\ lbf \cdot in$<- LL controls

Design shear from wind on rail: $V_{rail\_wind\_u} := \gamma_{WS} \cdot w_{rail\_wind} \cdot \frac{L_{spc}}{2}$ $V_{rail\_wind\_u} = 0.079\ kip$

$V_{rail\_u} = 0.556\ kip$ <- LL controls

Base Plate Design - Line Post w/ Axial Compression

Given: *Plans*

Cap width: $W_{cap} := 15.63\ in$

Distance from post to end of cap: $L_{end} := 72\ in$

Plate thickness: $t_p := 0.5\ in$

Plate length (perpendicular to fence): $N_{plate} := 8\ in$

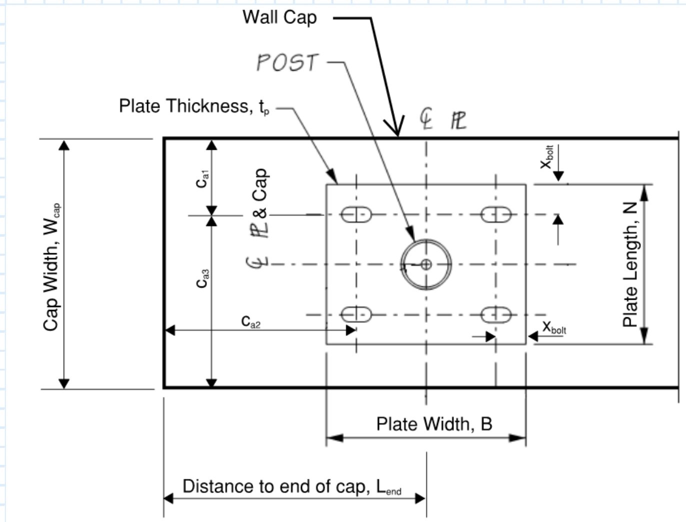
Plate width (parallel to fence): $B_{plate} := 10\ in$

Compressive Strength of Concrete: $f'_c := 4\ ksi$

Side clearance to anchor bolts: $x_{bolt} := 1.5\ in$

Base plate steel yield strength: $F_{y\_plate} := 36\ ksi$

Number of rails: $n_{rail} := 2$



Output:

Plate Area: $A_{plate} := N_{plate} \cdot B_{plate}$ $A_{plate} = 80\ in^2$

Distance from bolt to near face of cap: $c_{a1} := \frac{1}{2}\left(W_{cap} - N_{plate}\right) + x_{bolt}$ $c_{a1} = 5.315\ in$

Distance from outside bolt to end of cap: $c_{a2} := L_{end} - \frac{B_{plate}}{2} + x_{bolt}$ $c_{a2} = 68.5\ in$

Distance from bolt to far face of cap: $c_{a3} := W_{cap} - c_{a1}$ $c_{a3} = 10.315\ in$

Bearing Area taken to Be Same as Plate Area: $A_{bearing} := A_{plate}$ $A_{bearing} = 80\ in^2$ *Conservatively setting bearing area to the same as the plate.*

Max allowed bearing pressure: $f_{pu\_max} := \phi_{brg} \cdot min\left(0.85 \cdot f'_c \cdot \sqrt{\frac{A_{bearing}}{A_{plate}}}, 1.7 \cdot f'_c\right)$ $f_{pu\_max} = 2.04\ ksi$ *ACI Tbl. 14.5.6.1*

Max allowed bearing pressure line: $q_{max} := f_{pu\_max} \cdot B_{plate}$ $q_{max} = \left(2.448 \cdot 10^5\right) \frac{lbf}{ft}$

Post dead load on plate $P_{post\_DL} := w_{post} \cdot H_{post}$ $P_{post\_DL} = 0.016\ kip$

Rail dead load on plat: $P_{rail\_DL} := n_{rail} \cdot 2 \cdot V_{rail\_DL}$ $P_{rail\_DL} = 0.032\ kip$

Factored vertical load on plate: $P_u := \gamma_{DL} \cdot \left(P_{post\_DL} + P_{rail\_DL}\right)$ $P_u = 0.06\ kip$

Minimum length of area of bearing: $Y_{min} := \frac{P_u}{q_{max}}$ $Y_{min} = 0.003\ in$ *AISC DG#1 Eqn. 3.3.3*

Critical eccentricity distance: $e_{crit} := \frac{N_{plate}}{2} - \frac{Y_{min}}{2}$ $e_{crit} = 3.999\ in$ *AISC DG#1 Eqn. 3.3.7*

Eccentricity of loading: $e_{loading} := \frac{M_{post\_u}}{P_u}$ $e_{loading} = 607.565\ in$ *AISC DG#1 Eqn. 3.3.6*

Small moment check: $Small\_Moment\_Check :=$ if $e_{loading} \le e_{crit}$
  "Moment is small, no need for anchor bolts."
else
  "Moment is large, need anchor bolts."

$Small\_Moment\_Check =$ "Moment is large, need anchor bolts."

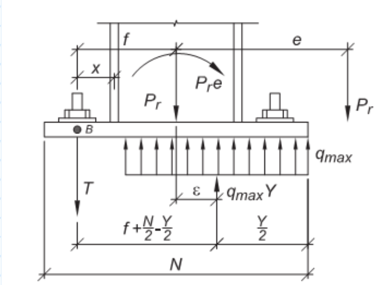


Fig. 3.4.1. Base plate with large moment.

Distance from bolt to center of post: $f_{dim} := \frac{N_{plate}}{2} - x_{bolt}$ $f_{dim} = 2.5\ in$

Plate dimension check

$$Plate\_Dim\_Check := \text{if } \left(f_{dim} + \frac{N_{plate}}{2}\right)^2 \ge \frac{2 \cdot P_u \cdot \left(e_{loading} + f_{dim}\right)}{q_{max}}$$
$$\quad \text{“Plate dimensions are OK.”}$$
$$\text{else}$$
$$\quad \text{“Plate needs to be longer and/or wider.”}$$

$Plate\_Dim\_Check =$ “Plate dimensions are OK.”

Length of bearing area centered at the eccentricity of this loading:

$$Y_{loading} := \left(f_{dim} + \frac{N_{plate}}{2}\right) - \sqrt{\left(f_{dim} + \frac{N_{plate}}{2}\right)^2 - \frac{2 \cdot P_u \cdot \left(e_{loading} + f_{dim}\right)}{q_{max}}}$$

$Y_{loading} = 0.285$ ***in***

AISC DG#1 Eqn. 3.4.3

Required tensile resistance in anchor rods:

$$T_u := q_{max} \cdot Y_{loading} - P_u$$

$T_u = 5.744$ ***kip***

AISC DG#1 Eqn. 3.4.2

Find minimum required thickness for plate based on bending at bearing interface:

Find bearing bending line distance from edge of plate (AISC DG#1, 3.1.3):

$$m_{plate} := \frac{N_{plate} - 0.8\ OD_{post}}{2}$$

$m_{plate} = 2.85$ ***in***

Calculating minimum thickness based on bearing:

$$t_{p\_brng\_req} := \text{if } Y_{loading} \ge m_{plate}$$
$$\quad 1.5 \cdot m_{plate} \cdot \sqrt{\frac{f_{pu\_max}}{F_{y\_plate}}}$$
$$\text{else}$$
$$\quad 2.11 \cdot \sqrt{\frac{f_{pu\_max} \cdot Y_{loading} \cdot \left(m_{plate} - \frac{Y_{loading}}{2}\right)}{F_{y\_plate}}}$$

$t_{p\_brng\_req} = 0.441$ ***in***

AISC DG#1 Eqns. 3.3.14a-2 & 3.3.15a-2

Find minimum required thickness for plate based on bending at tension interface:

Find tension bending line distance from edge of plate (AISC DG#1, 3.1.3):

$$x_{ten} := f_{dim} - \frac{0.8 \cdot OD_{post}}{2}$$

$x_{ten} = 1.35$ ***in***

Calculating minimum thickness based on tension:

$$t_{p\_ten\_req} := 2.11 \cdot \sqrt{\frac{T_u \cdot x_{ten}}{B_{plate} \cdot F_{y\_plate}}}$$

$t_{p\_ten\_req} = 0.31$ ***in***

AISC DG#1 Eqn. 3.4.7a

Controlling minimum required base plate thickness:

$$t_{p\_req} := \max\left(t_{p\_brng\_req}, t_{p\_ten\_req}\right)$$

$t_{p\_req} = 0.441$ ***in***

Check chosen plate thickness:

$$Plate\_Thick\_Check := \text{if } t_p \ge t_{p\_req}$$
$$\quad \text{“Chosen plate thickness is adequate.”}$$
$$\text{else}$$
$$\quad \text{“Need a thicker plate.”}$$

$Plate\_Thick\_Check =$ “Chosen plate thickness is adequate.”

Pipe to Plate Fillet Weld Connection Design

Given:

Minimum Fillet Weld Size: $w_{min} := \frac{1}{8}\ in$ *Min fillet weld size based on AISC Table J2-4*

Chosen fillet weld size $w := \frac{5}{16}\ in$

Weld material: $F_{EXX} := 70\ ksi$

Output:

Welded Connection to Base Plate Design:

Gross Length of Weld is Post Perimeter: $L_g := \pi \cdot OD_{post}$ $L_g = 9.032\ in$

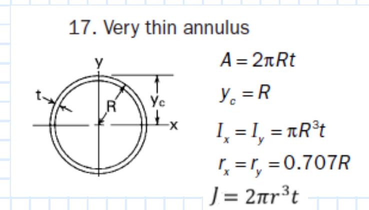
Effective Length of Weld: $L_w := L_g - 2 \cdot w$ $L_w = 8.407\ in$

Effective Throat Thickness: $t_e := min\left(w \cdot \sin(45\ deg), \frac{L_w}{4}\right)$ $t_e = 0.221\ in$ *AISC, Sect. J2, Pts. 2a & 2b*

Area of Weld: $A_w := L_w \cdot t_e$ $A_w = 1.858\ in^2$ *AISC, Sect. J2, Pts. 2a*

Moment of Inertia of circular fillet weld: $I_w := \pi \cdot \left(\frac{OD_{post}}{2}\right)^3 \cdot t_e$ $I_w = 2.062\ in^4$

Polar moment of Inertia of circular fillet weld: $J_w := 2\ \pi \cdot \left(\frac{OD_{post}}{2}\right)^3 \cdot t_e$ $J_w = 4.124\ in^4$



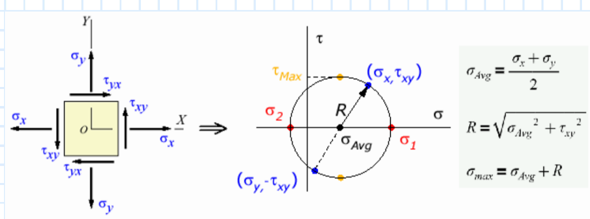
Determine design strength of weld:

Nominal strength of weld metal: $F_w := \phi_{fw} \cdot 0.6 \cdot F_{EXX}$ $F_w = 31.5\ ksi$ *AISC, Tbl. J2.5*

Normal stress caused by bending moment: $\sigma_b := \frac{M_{post\_u} \cdot \left(\frac{OD_{post}}{2}\right)}{I_w}$ $\sigma_b = 25.619\ ksi$ $\sigma = \frac{M}{S} = \frac{M \cdot c}{I}$

Stress caused by shearing force: $\tau_v := \frac{V_{post\_u}}{A_w}$ $\tau_v = 0.471\ ksi$

Resultant stress in weld from loading: $\sigma_{max} := \frac{\sigma_b}{2} + \sqrt{\left(\frac{\sigma_b}{2}\right)^2 + {\tau_v}^2}$ $\sigma_{max} = 25.627\ ksi$

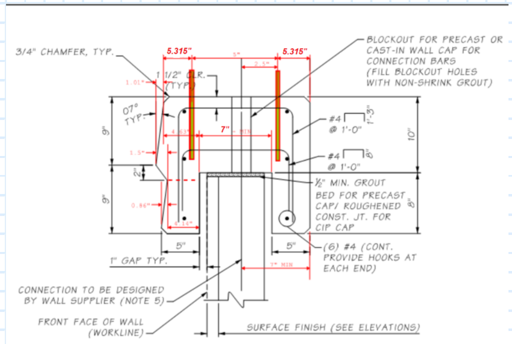


Check of weld thickness:

$Weld\_Design\_Check :=$ if $F_w \geq \sigma_{max}$
  "Chosen weld size is sufficient."
else
  "Need bigger fillet weld."

$Weld\_Design\_Check =$ "Chosen weld size is sufficient."

Anchor Bolt Connection Design



WALL CAP TYPICAL DETAIL

Given:

Number of anchor bolts resisting loads: $n_{ab} := 2$ *Only one side's bolts resist tension or shear.*

Bolts are specified as ASTM F1554 and Grade A36

Bolt diameter: $d_{ab} := \frac{5}{8}\ in$ *Plans*

Bolt area: $A_b := 0.307\ in^2$ *AISC Tbl. 7-18*

Bolt nominal yield stress strength: $F_{y\_bolt} := 36\ ksi$ *AISC Tbl. 2-3*

Bolt nominal ultimate tensile stress strength: $F_{u\_bolt} := 58\ ksi$

Bolt embedment: $h_{ef} := 5\ in$ *Plans*

Output:

Tension anchor bolt spacing: $s_1 := \frac{B_{plate} - 2 \cdot x_{bolt}}{n_{ab} - 1}$ $s_1 = 7\ in$

Bolt nominal tensile stress strength: $F_{nt} := 0.75 \cdot F_{u\_bolt}$ $F_{nt} = 43.5\ ksi$ *AISC Tbl. J3.2*

Bolt nominal shear stress strength: $F_{nv} := 0.40 \cdot F_{u\_bolt}$ $F_{nv} = 23.2\ ksi$ *AISC Tbl. J3.2, assuming threads within shear plane*

Ultimate tension load on one anchor bolt: $T_{u\_ab} := \frac{T_u}{n_{ab}}$ $T_{u\_ab} = 2.872\ kip$

Required shear stress on one bolt: $f_v := \frac{V_{post\_u}}{n_{ab} \cdot A_b}$ $f_v = 1.425\ ksi$

Bolt modified nominal tensile stress strength, modified for effects of shear stress: $F_{nt}' := min\left(F_{nt}, 1.3 \cdot F_{nt} - \frac{F_{nt}}{\phi_{ab} \cdot F_{nv}} \cdot f_v\right)$ $F_{nt}' = 43.5\ ksi$ *AISC Eqn. J3-3a*

Bolt factored tensile resistance: $\phi R_{n\_bolt} := \phi_{ab} \cdot F_{nt}' \cdot A_b$ $\phi R_{n\_bolt} = 10.016\ kip$ *AISC Eqn. J3-2*

Check of bolt tensile stress:

$Bolt\_Tensile\_Check :=$ if $\phi R_{n\_bolt} \ge T_{u\_ab}$
  "Bolt is satisfactory."
else
  "Bolt is no good."

$Bolt\_Tensile\_Check =$ "Bolt is satisfactory."

Continuing Anchor Bolt Connection Design per ACI 318

Outside diameter of anchor: $d_a := d_{ab}$ $d_a = 0.625\ in$

Critical edge distance for adhesive anchors: $c_{ac} := 2\ h_{ef}$ $c_{ac} = 10\ in$ *ACI 17.7.6*

Steel strength of anchor in tension (ACI 17.4.1)

Steel tension strength of anchor is confirmed above; so, no check here is necessary.

Concrete breakout strength of anchor in tension (ACI 17.4.2)

Check bolt group action for tension concrete breakout: *ACI 17.2.1.1*

$Group\_Tension\_Breakout\_Check :=$ if $s_1 \le 3 \cdot h_{ef}$
  "Bolts act in group."
else
  "Bolts act singly."

$Group\_Tension\_Breakout\_Check =$ "Bolts act in group."

Theoretical projected influence area of a single bolt far from an edge: $A_{Nco} := 9 \cdot h_{ef}^2$ $A_{Nco} = 225\ in^2$ *ACI Eqn. 17.4.2.1c*

Actual projected influence area for bolt(s): $A_{Nc} := min\left(\left(c_{a1} + 1.5 \cdot h_{ef}\right) \cdot \left(1.5 \cdot h_{ef} + min\left(s_1, 3 \cdot h_{ef}\right) + min\left(1.5 \cdot h_{ef}, c_{a2}\right)\right), n_{ab} \cdot A_{Nco}\right)$ $A_{Nc} = 281.93\ in^2$ *ACI Fig. R17.4.2.1*

Concrete tension breakout strength coefficient: $k_c := 17$ *Value of 17 for post-installed anchors, per ACI 17.4.2.2*

Basic concrete tension breakout strength for single anchor: $N_b := k_c \cdot 1.0 \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{h_{ef}}{in}\right)^{1.5} \cdot lbf$ $N_b = 12.021\ kip$ *ACI Eqn. 17.4.2.2a*

Factor for eccentrically loaded anchor bolts: $\Psi_{ec\_N} := 1.0$ *Anchor bolts are not loaded eccentrically.* *ACI 17.4.2.4*

Factor for anchor bolts near an edge: $\Psi_{ed\_N} := min\left(1.0, 0.7 + 0.3 \cdot \frac{c_{a1}}{1.5 \cdot h_{ef}}\right)$ $\Psi_{ed\_N} = 0.913$ *ACI Eqn. 17.4.2.5b*

Factor for anchor bolts in un-cracked concrete: $\Psi_{c\_N} := 1.4$ *Wall caps are not under load, and per the wall cap design, service moment from post does not cause cracking.* *ACI 17.4.2.6*

Factor for anchor bolts in un-cracked concrete near an edge without supplementary reinforcement: $\Psi_{cp\_N} := min\left(1.0, \max\left(\frac{c_{a1}}{c_{ac}}, \frac{1.5 \cdot h_{ef}}{c_{ac}}\right)\right)$ $\Psi_{cp\_N} = 0.75$ *ACI Eqn. 17.4.2.7b*

Nominal concrete tension breakout strength: $\phi N_{cbg} := \phi_{adh} \cdot \frac{A_{Nc}}{A_{Nco}} \cdot \Psi_{ec\_N} \cdot \Psi_{ed\_N} \cdot \Psi_{c\_N} \cdot \Psi_{cp\_N} \cdot N_b$ $\phi N_{cbg} = 9.382\ kip$ *ACI Eqn. 17.4.2.1b*

Check of concrete tension breakout failure:

$$Concrete\_Tension\_Breakout\_Check := \begin{cases} \text{"Bolt is satisfactory."} & \text{if } \phi N_{cbg} \ge n_{ab} \cdot T_{u\_ab} \\ \text{"Bolt is no good."} & \text{else} \end{cases}$$

$Concrete\_Tension\_Breakout\_Check =$ "Bolt is satisfactory."

Pullout strength cast-in, post-installed expansion, or undercut anchor in tension (ACI 17.4.3)

Proposed anchors are post-installed adhesive, not headed studs or bolts, expansion anchors, or undercut anchors; so, no check is required.

Concrete side-face blowout strength of headed anchor in tension (ACI 17.4.4)

Proposed anchors are post-installed adhesive, not headed studs or bolts; so, no check is required.

Bond strength of adhesive anchor in tension (ACI 17.4.5)

Minimum bond stress for HY 200 Epoxy per HILTI ESR-3187:

$$\tau_{uncr\_HY\_200} := 0.65 \cdot \left(\frac{f'_c}{2500\ psi}\right)^{0.1} \cdot 2220\ psi = 1512.441\ psi$$

Per HILTI ESR-3187 Table 14, basic un-cracked bond strength is 2,220 psi; this value is factored by a straight 0.65 for either wet or dry installation conditions and by a small boost from concrete strength higher than 2,500 psi

Minimum bond stress for HIT-RE 500 Epoxy per HILTI ESR-3814:

$$\tau_{uncr\_HIT\_RE\_500} := 0.65 \cdot \left(\frac{f'_c}{2500\ psi}\right)^{0.15} \cdot 2210\ psi = 1541.429\ psi$$

Per HILTI ESR-3814 Table 12, basic un-cracked bond strength is 2,210 psi. This value is based on diamond coring and roughening afterwards; it is lower than being hammer-drilled with carbide bit. The socket must be roughened if coring with a diamond bit; this should be written on the plans. Factors are a straight 0.65 reduction factor independent of wet or dry concrete conditions during installation and a small boost for using concrete higher than 2,500 psi. The smaller factor for cracked concrete is used since no supplementary rebar is being provided; this also matches with reduction factor below.

Minimum bond stress strength: $\tau_{uncr} := min\left(\tau_{uncr\_HY\_200}, \tau_{uncr\_HIT\_RE\_500}\right)$ $\tau_{uncr} = 1512.441\ psi$

Distance to edge of project influence area: $c_{Na} := 10 \cdot d_a \cdot \sqrt{\frac{\tau_{uncr}}{1100\ psi}}$ $c_{Na} = 7.329\ in$ *ACI Eqn. 17.4.5.1d*

Check if anchor bolts act in group for bond failure: *ACI 17.2.1.1*

$$Group\_Bond\_Failure\_Check := \begin{cases} \text{"Bolts act in group."} & \text{if } s_1 \le 2\ c_{Na} \\ \text{"Bolts act singly."} & \text{else} \end{cases}$$

$Group\_Bond\_Failure\_Check =$ "Bolts act in group."

Theoretical projected influence area of a single bolt far from an edge: $A_{Nao} := (2\ c_{Na})^2$ $\quad A_{Nao} = 214.835\ \mathbf{in}^2$ *ACI Eqn. 17.4.5.1c*

Actual projected influence area for bolt(s): $A_{Na} := min\left(\left(c_{Na} + min\left(s_1, 2 \cdot c_{Na}\right) + min\left(c_{Na}, c_{a2}\right)\right) \cdot \left(c_{a1} + c_{Na}\right), n_{ab} \cdot A_{Nao}\right)$ $\quad A_{Na} = 273.826\ \mathbf{in}^2$ *ACI Fig. R17.4.5.1*

Basic bond strength of adhesive anchor: $N_{ba} := \tau_{uncr} \cdot \pi \cdot d_a \cdot h_{ef}$ $\quad N_{ba} = 14.848\ \mathbf{kip}$ *ACI Eqn. 17.4.5.2*

Concrete is not light weight; so, lambda-a is set to 1.0; per ACI 17.4.5.2, un-cracked bond stress may be used.

Factor for eccentrically loaded anchor bolts: $\Psi_{ec\_Na} := 1.0$ *Anchor bolts are not loaded eccentrically.* *ACI 17.4.5.3*

Factor for anchor bolts near an edge: $\Psi_{ed\_Na} := min\left(1.0, 0.7 + 0.3 \cdot \dfrac{c_{a1}}{c_{Na}}\right)$ $\quad \Psi_{ed\_Na} = 0.918$ *ACI Eqn. 17.4.5.4b*

Factor for anchor bolts in un-cracked concrete near an edge without supplementary reinforcement: $\Psi_{cp\_Na} := min\left(1.0, \max\left(\dfrac{c_{a1}}{c_{ac}}, \dfrac{c_{Na}}{c_{ac}}\right)\right)$ $\quad \Psi_{cp\_Na} = 0.733$ *ACI Eqn. 17.4.5.5b*

Wall caps are not under load, and per the wall cap design, service moment from post does not cause cracking.

Nominal bond strength of the adhesive anchor(s): $\phi N_{ag} := \phi_{adh} \cdot \dfrac{A_{Na}}{A_{Nao}} \cdot \Psi_{ec\_Na} \cdot \Psi_{ed\_Na} \cdot \Psi_{cp\_Na} \cdot N_{ba}$ $\quad \phi N_{ag} = 8.272\ \mathbf{kip}$ *ACI Eqn. 17.4.5.1b*

Check of bolt bond stress failure:

```
Bond_Stress_Check := if ϕN_ag ≥ n_ab • T_u_ab
                         “Bolt is satisfactory.”
                     else
                         “Bolt is no good.”
```

$Bond\_Stress\_Check =$ “Bolt is satisfactory.”

Steel strength of anchor in shear (17.5.1)

Steel shear strength of anchor is confirmed above; so, no check here is necessary.

Concrete breakout strength of anchor in shear (17.5.2)

Check bolt group action for shear concrete breakout: *ACI 17.2.1.1*

```
Group_Shear_Breakout_Check := if s_1 ≤ 3 • c_a1
                                  “Bolts act in group.”
                              else
                                  “Bolts act singly.”
```

$Group\_Shear\_Breakout\_Check =$ “Bolts act in group.”

Theoretical projected influence area of a single bolt far from an edge: $A_{Vco} := 4.5 \cdot {c_{a1}}^2$ $\quad A_{Vco} = 127.122\ \mathbf{in}^2$ *ACI Eqn. 17.5.2.1c*

Actual projected influence area for bolt(s): $A_{Vc} := min\left(1.5 \cdot c_{a1} \cdot \left(1.5 \cdot c_{a1} + min\left(s_1, 3 \cdot c_{a1}\right) + min\left(1.5 \cdot c_{a1}, c_{a2}\right)\right), n_{ab} \cdot A_{Vco}\right)$ $\quad A_{Vc} = 182.929\ \mathbf{in}^2$ *ACI Fig. R17.5.2.1b*

Load bearing length: $l_e := h_{ef}$ $\quad l_e = 5\ \mathbf{in}$ *ACI 17.5.2.2*

Basic concrete breakout strength in shear for single anchor:

$$V_b := min\left(\left(7\left(\frac{l_e}{d_a}\right)^{0.2} \cdot \sqrt{\frac{d_a}{\mathbf{in}}}\right) \cdot 1.0 \cdot \sqrt{\frac{f'_c}{\mathbf{psi}}} \cdot \left(\frac{c_{a1}}{\mathbf{in}}\right)^{1.5}, 9 \cdot 1.0 \cdot \sqrt{\frac{f'_c}{\mathbf{psi}}} \cdot \left(\frac{c_{a1}}{\mathbf{in}}\right)^{1.5}\right) \cdot \mathbf{lbf} \qquad V_b = 6.5\ \mathbf{kip}$$

Concrete is not light weight; so, lambda-a is set to 1.0. *ACI Eqns. 17.5.2.2a & 17.5.2.2b*

Factor for eccentrically loaded anchor bolts: $\Psi_{ec\_V} := 1.0$ *Anchor bolts are not loaded eccentrically.* *ACI 17.5.2.5*

Factor for anchor bolts near an edge: $\Psi_{ed\_V} := min\left(1.0, 0.7 + 0.3 \cdot \dfrac{c_{a2}}{1.5 \cdot c_{a1}}\right)$ $\quad \Psi_{ed\_V} = 1$ *ACI Eqns. 17.5.2.6a & 17.5.2.6b*

Factor for anchor bolts in un-cracked concrete: $\Psi_{c\_V} := 1.4$ *Wall caps are not under load, and per the wall cap design, service moment from post does not cause cracking.* *ACI 17.5.2.7*

Factor for small embedment $\Psi_{h\_V} := min\left(1.0, \sqrt{\frac{1.5 \cdot c_{a1}}{h_{ef}}}\right)$ $\Psi_{h\_V} = 1$ *ACI Eqn. 17.5.2.8*

Nominal concrete shear breakout strength: $\phi V_{cbg} := \phi_{adh} \cdot \frac{A_{Vc}}{A_{Vco}} \cdot \Psi_{ec\_V} \cdot \Psi_{ed\_V} \cdot \Psi_{c\_V} \cdot \Psi_{h\_V} \cdot V_b$ $\phi V_{cbg} = 8.512$ ***kip*** *ACI Eqn. 17.5.2.1b*

Check of concrete shear breakout failure: *Concrete_Shear_Breakout_Check* := if $\phi V_{cbg} \geq V_{post\_u}$
‖ "Bolt is satisfactory."
else
‖ "Bolt is no good."

Concrete_Shear_Breakout_Check = "Bolt is satisfactory."

Concrete pryout strength of anchor in shear (17.5.3)

Basic concrete pryout strength of a single anchor in shear: $\phi N_{cpg} := min\left(\phi N_{ag}, \phi N_{cbg}\right)$ $\phi N_{cpg} = 8.272$ ***kip*** *ACI 17.5.3.1*

Concrete pryout strength in shear coefficient: $k_{cp} :=$ if $h_{ef} < 2.5$ ***in***
‖ 1.0
else
‖ 2.0

$k_{cp} = 2$ *ACI 17.5.3.1*

Nominal concrete pryout strength of anchor(s) in shear: $\phi V_{cpg} := k_{cp} \cdot \phi N_{cpg}$ $\phi V_{cpg} = 16.545$ ***kip*** *ACI Eqn. 17.5.3.1b*

Check of concrete pryout strength in shear: *Concrete_Shear_Pryout_Check* := if $\phi V_{cpg} \geq V_{post\_u}$
‖ "Bolt is satisfactory."
else
‖ "Bolt is no good."

Concrete_Shear_Pryout_Check = "Bolt is satisfactory."

Vertical Interior Post and Horizontal Rail Design

Given:

Post Height: $H_{post} := 66\ in$ *Plans*

Step Height: $H_{step} := 24\ in$

Post and Rail Yield Strength: $F_y := 50\ ksi$ *ASTM F1043*

Post and Rail Modulus of Elasticity: $E_s := 29000\ ksi$

Post and Rail Ultimate Strength: $F_u := 58\ ksi$

Properties for ASTM F1043 IC 2-7/8" Pipe for Interior Posts

Post OD: $OD_{post} := 2.875\ in$

Post Thickness: $t_{post} := 0.160\ in$

Properties for ASTM F1043 IC 1-5/8" Pipe for Rails

Rail OD: $OD_{rail} := 1.660\ in$

Rail Thickness: $t_{rail} := 0.111\ in$

Design Point Live Load $P_{LL} := 200\ lbf$ *AASHTO 13.8.2*

Design Uniform Live Load $w_{LL} := 50\ plf$ *AASHTO 13.8.2*

Post spacing: $L_{spc} := 4\ ft$ *Plans*

Weight of chain link fence: $f_{clf} := 0.48\ psf$

Design wind load from chain link fence: $f_{wind} := 15\ psf$ *AASHTO 13.8.2*

Load Factors (AASHTO Tbl. 3.4.1-1):

PL Load Factor: $\gamma_{PL} := 1.75$

DC Load Factor: $\gamma_{DL} := 1.25$

WS Load Factor: $\gamma_{WS} := 1.00$

Resistance Factors:

Steel Flexure (AASHTO 6.5.4.2): $\phi_f := 1.00$

Steel Shear (AASHTO 6.5.4.2): $\phi_v := 1.00$

Tension, Yielding in Gross Section: $\phi_y := 0.95$

Bending (AISC F1): $\phi_b := 0.90$

Shear (AISC G1): $\phi_{v\_AISC} := 0.90$

Bearing (AISC DG#1): $\phi_{brg} := 0.60$

Fillet Weld (AISC Tbl. J2.5): $\phi_{fw} := 0.75$

Bolts (AISC J3.6 & J3.7): $\phi_{ab} := 0.75$

Adhesive Anchor Bolts (ACI 17.3.3, Condition B, Category 1): $\phi_{adh} := 0.65$

Steel weight density: $\gamma_{steel} := 490\ pcf$

ASTM F1043 Group IC Electric Resistant Welded 50,000 psi yield steel pipe

Trade Reference	Decimal O.D. Equivalent		Pipe wall Thickness		Weight		Section Modulus		x	Min. Yield Strength		=	Max Bending Moment	Calculated Load (lbs)		
														10' Free	Cantilever	
O.D.	inches	(mm)	inches	(mm)	lb./ft.	(kg/m)	inches³	(mm³)	x	psi	(Mpa)	=	Lb.In.	Supported	4'	6'
1 5/8"	1.660	42.16	0.111	2.82	1.84	2.74	0.1962	4.98	x	50000	345	=	9810	327	204	136
1 7/8"	1.900	48.26	0.120	3.05	2.28	3.39	0.2810	7.14	x	50000	345	=	14050	468	293	195
2 3/8"	2.375	60.33	0.130	3.30	3.12	4.64	0.4881	12.40	x	50000	345	=	24405	814	508	339
2 7/8"	2.875	73.03	0.160	4.06	4.64	6.91	0.8778	22.30	x	50000	345	=	43890	1463	914	610
3 1/2"	3.500	88.90	0.160	4.06	5.71	8.50	1.3408	34.06	x	50000	345	=	67042	2235	1397	931
4"	4.000	101.60	0.160	4.06	6.57	9.78	1.7820	45.26	x	50000	345	=	89098	2970	1856	1237
4 1/2"	4.500	114.30	0.160	4.14	7.42	11.04	2.2859	57.99	X	50000	345	=	114295	3810	5486	1587

Output:

Post Section Properties:

Post inside diameter: $ID_{post} := OD_{post} - 2 \cdot t_{post}$ $ID_{post} = 2.555\ in$

Post Area: $A_{post} := 0.785398 \cdot \left(OD_{post}^2 - ID_{post}^2\right)$ $A_{post} = 1.365\ in^2$

Post Unit Weight: $w_{post} := \gamma_{steel} \cdot A_{post}$ $w_{post} = 4.644\ plf$

Post centroid: $c_{post} := 0.5 \cdot OD_{post}$ $c_{post} = 1.438\ in$

Post Moment of Inertial: $I_{post} := 0.049087 \cdot \left(OD_{post}^4 - ID_{post}^4\right)$ $I_{post} = 1.262\ in^4$

Post Section Modulus: $S_{post} := \dfrac{I_{post}}{c_{post}}$ $S_{post} = 0.878\ in^3$

Post Plastic Section Modulus: $Z_{post} := \dfrac{OD_{post}^3 - ID_{post}^3}{6}$ $Z_{post} = 1.181\ in^3$

Rail Section Properties:

Rail inside diameter: $ID_{rail} := OD_{rail} - 2 \cdot t_{rail}$ $ID_{rail} = 1.438\ in$

Rail Area: $A_{rail} := 0.785398 \cdot \left(OD_{rail}^2 - ID_{rail}^2\right)$ $A_{rail} = 0.54\ in^2$

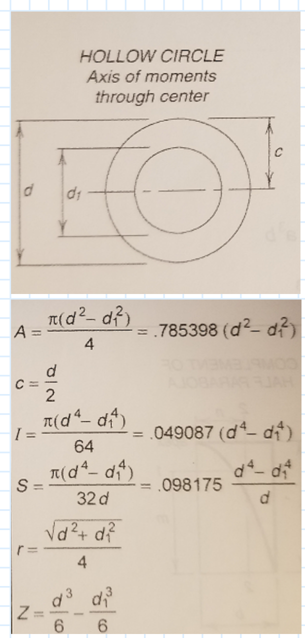
Rail Unit Weight: $w_{rail} := \gamma_{steel} \cdot A_{rail}$ $w_{rail} = 1.838\ plf$

Rail centroid: $c_{rail} := 0.5 \cdot OD_{rail}$ $c_{rail} = 0.83\ in$

Rail Moment of Inertial: $I_{rail} := 0.049087 \cdot \left(OD_{rail}^4 - ID_{rail}^4\right)$ $I_{rail} = 0.163\ in^4$

Rail Section Modulus: $S_{rail} := \dfrac{I_{rail}}{c_{rail}}$ $S_{rail} = 0.196\ in^3$

Rail Plastic Section Modulus: $Z_{rail} := \dfrac{OD_{rail}^3 - ID_{rail}^3}{6}$ $Z_{rail} = 0.267\ in^3$



Post concentrated live load applied at high top rail: $P_{post\_LL\_H} := P_{LL} + w_{LL} \cdot \frac{L_{spc}}{2} = 0.3\ kip$ $P_{post\_LL\_H} = 0.3\ kip$ *AASHTO Eqn. 13.8.2-1, modified for split top rails*

Post concentrated live load applied at low top rail: $P_{post\_LL\_L} := w_{LL} \cdot \frac{L_{spc}}{2} = 0.1\ kip$ $P_{post\_LL\_L} = 0.1\ kip$

Post moment loading from live load: $M_{post\_LL} := P_{post\_LL\_H} \cdot H_{post} + P_{post\_LL\_L} \cdot (H_{post} - H_{step})$ $M_{post\_LL} = 24000\ lbf \cdot in$ *Post treated as cantilevered beam*

Post shear from live load: $V_{post\_LL} := P_{post\_LL\_H} + P_{post\_LL\_L}$ $V_{post\_LL} = 0.4\ kip$

Rail moment from live load applied: $M_{rail\_LL} := \sqrt{2} \cdot \frac{w_{LL} \cdot L_{spc}^{\ 2}}{8} + \frac{P_{LL} \cdot L_{spc}}{4}$ $M_{rail\_LL} = 4097.056\ lbf \cdot in$ *Rail treated as simply supported beam with vertical and horizontal live loads combined into resultant direction.*

Rail moment from dead load: $M_{rail\_DL} := \frac{w_{rail} \cdot L_{spc}^{\ 2}}{8} + \frac{f_{clf} \cdot \frac{H_{post}}{2} \cdot L_{spc}^{\ 2}}{8}$ $M_{rail\_DL} = 75.793\ lbf \cdot in$

Rail shear from live load: $V_{rail\_LL} := \sqrt{2} \cdot \frac{w_{LL} \cdot L_{spc}}{2} + \frac{P_{LL}}{2}$ $V_{rail\_LL} = 0.241\ kip$

Rail shear from dead load: $V_{rail\_DL} := \frac{w_{rail} \cdot L_{spc}}{2} + \frac{f_{clf} \cdot \frac{H_{post}}{2} \cdot L_{spc}}{2}$ $V_{rail\_DL} = 0.006\ kip$

Factored Shear Load on Post: $V_{post\_u} := \gamma_{PL} \cdot V_{post\_LL}$ $V_{post\_u} = 0.7\ kip$

Factored Moment Load on Post: $M_{post\_u} := \gamma_{PL} \cdot M_{post\_LL}$ $M_{post\_u} = 42000\ lbf \cdot in$

AASHTO load factors used instead of ASCE load factors found in AISC and ACI. This is acceptable as it is more conservative.

Factored Shear Load on Rail: $V_{rail\_u} := \gamma_{PL} \cdot V_{rail\_LL} + \gamma_{DL} \cdot V_{rail\_DL}$ $V_{rail\_u} = 0.43\ kip$

Factored Moment Load on Rail: $M_{rail\_u} := \gamma_{PL} \cdot M_{rail\_LL} + \gamma_{DL} \cdot M_{rail\_DL}$ $M_{rail\_u} = 7264.59\ lbf \cdot in$

Vertical dead load was combined directly with live load resultant since it was so small compared to the live load.

Post Analysis:

Following AASHTO 6.12.1.2.3c for Shear Design:

Gross Area: $A_g := A_{post}$ $A_g = 1.365\ in^2$

Distance from Max to 0 Shear: $L_v := H_{post}$ $L_v = 66\ in$

Critical Strength for Shear:

$$F_{cr} := min\left(0.58 \cdot F_y, \max\left(\frac{1.6 \cdot E_s}{\sqrt{\frac{L_v}{OD_{post}}}\left(\frac{OD_{post}}{t_{post}}\right)^{\frac{5}{4}}}, \frac{0.78 \cdot E_s}{\left(\frac{OD_{post}}{t_{post}}\right)^{\frac{3}{2}}}\right)\right)$$

$F_{cr} = 29\ ksi$ *AASHTO Eqns. 6.12.1.2.3c-2 & 6.12.1.2.3c-3*

Factored nominal shear resistance: $\phi V_n := \phi_v \cdot 0.5\ F_{cr} \cdot A_g$ $\phi V_n = 19.788\ kip$ *AASHTO Eqn. 6.12.1.2.3c-1*

Post Shear Check: $\frac{\phi V_n}{V_{post\_u}} = 28.269$

$Post\_Shear\_Check :=$ if $\frac{\phi V_n}{V_{post\_u}} \geq 1.0$
  "Post shear strength is satisfactory."
else
  "Post is not satisfactory."

$Post\_Shear\_Check =$ "Post shear strength is satisfactory."

Following AASHTO 6.12.2.2.3 for Flexure Design:

Check of Noncompact Section:

$Check\_Compact :=$ if $\frac{OD_{post}}{t_{post}} \leq \frac{0.07 \cdot E_s}{F_y}$
  "Section is compact. Local buckling does not apply."
else
  "Section is not compact. Check wall slenderness."

Per AASHTO 6.12.2.2.3, as long D/t does not exceed 0.07E/Fy, plastic modulus and equation 6.12.2.2.3-1 may be used.

$Check\_Compact =$ "Section is compact. Local buckling does not apply."

Factored Nominal Moment Resistance: $\phi M_n := \phi_f \cdot F_y \cdot Z_{post}$ $\phi M_n = 59.038\ kip \cdot in$ AASHTO Eqn. 6.12.2.2.3-1

Post Flexural Check: $\dfrac{\phi M_n}{M_{post\_u}} = 1.406$

$Post\_Flex\_Check :=$ if $\dfrac{\phi M_n}{M_{post\_u}} \geq 1.0$
"Post flexural strength is satisfactory."
else
"Post is not satisfactory."

$Post\_Flex\_Check =$ "Post flexural strength is satisfactory."

Rail Analysis:

Following AASHTO 6.12.1.2.3c for Shear Design:

Gross Area: $A_g := A_{rail}$ $A_g = 0.54\ in^2$

Distance from Max to 0 Shear: $L_v := \dfrac{L_{spc}}{2}$ $L_v = 24\ in$

Critical Strength for Shear:

$$F_{cr} := min\left(0.58 \cdot F_y, \max\left(\frac{1.6 \cdot E_s}{\sqrt{\frac{L_v}{OD_{rail}}}\left(\frac{OD_{rail}}{t_{rail}}\right)^{\frac{5}{4}}}, \frac{0.78 \cdot E_s}{\left(\frac{OD_{rail}}{t_{rail}}\right)^{\frac{3}{2}}}\right)\right)$$

$F_{cr} = 29\ ksi$ AASHTO Eqns. 6.12.1.2.3c-2 & 6.12.1.2.3c-3

Factored Nominal Shear Resistance: $\phi V_n := \phi_v \cdot 0.5\ F_{cr} \cdot A_g$ $\phi V_n = 7.832\ kip$ AASHTO Eqn. 6.12.1.2.3c-1

Rail Shear Check: $\dfrac{\phi V_n}{V_{rail\_u}} = 18.199$

$Rail\_Shear\_Check :=$ if $\dfrac{\phi V_n}{V_{rail\_u}} \geq 1.0$
"Rail shear strength is satisfactory."
else
"Rail is not satisfactory."

$Rail\_Shear\_Check =$ "Rail shear strength is satisfactory."

Following AASHTO 6.12.2.2.3 for Flexure Design:

Check of Noncompact Section:

$Check\_Compact :=$ if $\dfrac{OD_{rail}}{t_{rail}} \leq \dfrac{0.07 \cdot E_s}{F_y}$
"Section is compact. Local buckling does not apply."
else
"Section is not compact. Check wall slenderness."

Per AASHTO 6.12.2.2.3, as long D/t does not exceed 0.07E/Fy, plastic modulus and equation 6.12.2.2.3-1 may be used.

$Check\_Compact =$ "Section is compact. Local buckling does not apply."

Factored Nominal Moment Resistance: $\phi M_n := \phi_f \cdot F_y \cdot Z_{rail}$ $\phi M_n = 13.339\ kip \cdot in$ AASHTO Eqn. 6.12.2.2.3-1

Post Flexural Check: $\dfrac{\phi M_n}{M_{rail\_u}} = 1.836$

$Rail\_Flex\_Check :=$ if $\dfrac{\phi M_n}{M_{rail\_u}} \geq 1.0$
"Rail flexural strength is satisfactory."
else
"Rail is not satisfactory."

$Rail\_Flex\_Check =$ "Rail flexural strength is satisfactory."

Confirming that Wind Loading Doesn't Control:

Per last paragraph of AASHTO 13.8.2, the wind load on the chain link fence is not applied simultaneously with the live load.

Uniform wind load on post: $w_{post\_wind} := f_{wind} \cdot L_{spc}$ $w_{post\_wind} = 60$ ***plf***

Design moment from wind on post: $M_{post\_wind\_u} := \gamma_{WS} \cdot \frac{w_{post\_wind} \cdot H_{post}^{2}}{2}$ $M_{post\_wind\_u} = 10890$ ***lbf • in***

$M_{post\_u} = 42000$ ***lbf • in*** <- LL controls

Design shear from wind on post: $V_{post\_wind\_u} := \gamma_{WS} \cdot w_{post\_wind} \cdot H_{post}$ $V_{post\_wind\_u} = 0.33$ ***kip***

$V_{post\_u} = 0.7$ ***kip*** <- LL controls

Uniform wind on rail: $w_{rail\_wind} := f_{wind} \cdot \frac{H_{post}}{2}$ $w_{rail\_wind} = 41.25$ ***plf***

Design moment from wind on rail: $M_{rail\_wind\_u} := \gamma_{WS} \cdot \frac{w_{rail\_wind} \cdot L_{spc}^{2}}{8}$ $M_{rail\_wind\_u} = 990$ ***lbf • in***

$M_{rail\_u} = 7264.59$ ***lbf • in*** <- LL controls

Design shear from wind on rail: $V_{rail\_wind\_u} := \gamma_{WS} \cdot w_{rail\_wind} \cdot \frac{L_{spc}}{2}$ $V_{rail\_wind\_u} = 0.083$ ***kip***

$V_{rail\_u} = 0.43$ ***kip*** <- LL controls

Base Plate Design - Line Post w/ Axial Compression

Given: *Plans*

Cap width: $W_{cap} := 15.63\ in$

Distance from post to end of cap: $L_{end} := 8\ in$

Plate thickness: $t_p := .5\ in$

Plate length (perpendicular to fence): $N_{plate} := 8\ in$

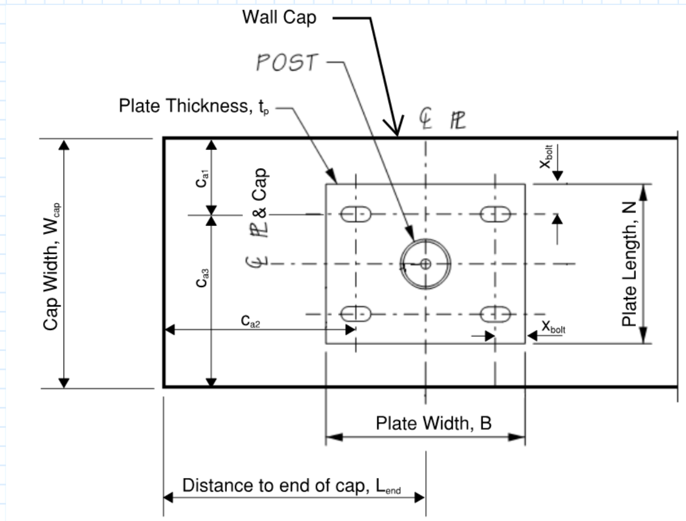
Plate width (parallel to fence): $B_{plate} := 10\ in$

Compressive Strength of Concrete: $f'_c := 4\ ksi$

Side clearance to anchor bolts: $x_{bolt} := 1.5\ in$

Base plate steel yield strength: $F_{y\_plate} := 36\ ksi$

Number of rails: $n_{rail} := 4$



Output:

Plate Area: $A_{plate} := N_{plate} \cdot B_{plate}$ $A_{plate} = 80\ in^2$

Distance from bolt to near face of cap: $c_{a1} := \frac{1}{2}\left(W_{cap} - N_{plate}\right) + x_{bolt}$ $c_{a1} = 5.315\ in$

Distance from outside bolt to end of cap: $c_{a2} := L_{end} - \frac{B_{plate}}{2} + x_{bolt}$ $c_{a2} = 4.5\ in$

Distance from bolt to far face of cap: $c_{a3} := W_{cap} - c_{a1}$ $c_{a3} = 10.315\ in$

Bearing Area taken to Be Same as Plate Area: $A_{bearing} := A_{plate}$ $A_{bearing} = 80\ in^2$ *Conservatively setting bearing area to the same as the plate.*

Max allowed bearing pressure: $f_{pu\_max} := \phi_{brg} \cdot min\left(0.85 \cdot f'_c \cdot \sqrt{\frac{A_{bearing}}{A_{plate}}}, 1.7 \cdot f'_c\right)$ $f_{pu\_max} = 2.04\ ksi$ *ACI Tbl. 14.5.6.1*

Max allowed bearing pressure line: $q_{max} := f_{pu\_max} \cdot B_{plate}$ $q_{max} = \left(2.448 \cdot 10^5\right) \frac{lbf}{ft}$

Post dead load on plate $P_{post\_DL} := w_{post} \cdot H_{post}$ $P_{post\_DL} = 0.026\ kip$

Rail dead load on plat: $P_{rail\_DL} := n_{rail} \cdot 2 \cdot V_{rail\_DL}$ $P_{rail\_DL} = 0.051\ kip$

Factored vertical load on plate: $P_u := \gamma_{DL} \cdot \left(P_{post\_DL} + P_{rail\_DL}\right)$ $P_u = 0.095\ kip$

Minimum length of area of bearing: $Y_{min} := \frac{P_u}{q_{max}}$ $Y_{min} = 0.005\ in$ *AISC DG#1 Eqn. 3.3.3*

Critical eccentricity distance: $e_{crit} := \frac{N_{plate}}{2} - \frac{Y_{min}}{2}$ $e_{crit} = 3.998\ in$ *AISC DG#1 Eqn. 3.3.7*

Eccentricity of loading: $e_{loading} := \frac{M_{post\_u}}{P_u}$ $e_{loading} = 441.7\ in$ *AISC DG#1 Eqn. 3.3.6*

Small moment check: $Small\_Moment\_Check :=$ if $e_{loading} \le e_{crit}$
  "Moment is small, no need for anchor bolts."
else
  "Moment is large, need anchor bolts."

$Small\_Moment\_Check =$ "Moment is large, need anchor bolts."

Distance from bolt to center of post: $f_{dim} := \frac{N_{plate}}{2} - x_{bolt}$ $f_{dim} = 2.5\ in$

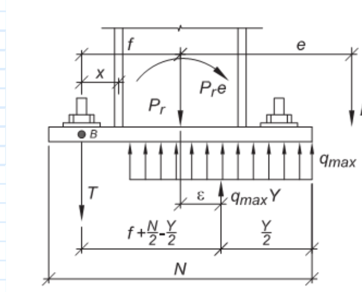


Fig. 3.4.1. Base plate with large moment.

Plate dimension check

$$Plate\_Dim\_Check := \text{if } \left(f_{dim} + \frac{N_{plate}}{2}\right)^2 \geq \frac{2 \cdot P_u \cdot \left(e_{loading} + f_{dim}\right)}{q_{max}} \; \Big\| \text{"Plate dimensions are OK."} \; \text{else} \; \Big\| \text{"Plate needs to be longer and/or wider."}$$

$Plate\_Dim\_Check =$ "Plate dimensions are OK."

Length of bearing area centered at the eccentricity of this loading:

$$Y_{loading} := \left(f_{dim} + \frac{N_{plate}}{2}\right) - \sqrt{\left(f_{dim} + \frac{N_{plate}}{2}\right)^2 - \frac{2 \cdot P_u \cdot \left(e_{loading} + f_{dim}\right)}{q_{max}}}$$

$Y_{loading} = 0.327\ in$

AISC DG#1 Eqn. 3.4.3

Required tensile resistance in anchor rods:

$$T_u := q_{max} \cdot Y_{loading} - P_u$$

$T_u = 6.571\ kip$ *AISC DG#1 Eqn. 3.4.2*

Find minimum required thickness for plate based on bending at bearing interface:

Find bearing bending line distance from edge of plate (AISC DG#1, 3.1.3):

$$m_{plate} := \frac{N_{plate} - 0.8\ OD_{post}}{2}$$

$m_{plate} = 2.85\ in$

Calculating minimum thickness based on bearing:

$$t_{p\_brng\_req} := \text{if } Y_{loading} \geq m_{plate} \; \Big\| 1.5 \cdot m_{plate} \cdot \sqrt{\frac{f_{pu\_max}}{F_{y\_plate}}} \; \text{else} \; \Big\| 2.11 \cdot \sqrt{\frac{f_{pu\_max} \cdot Y_{loading} \cdot \left(m_{plate} - \frac{Y_{loading}}{2}\right)}{F_{y\_plate}}}$$

$t_{p\_brng\_req} = 0.471\ in$

AISC DG#1 Eqns. 3.3.14a-2 & 3.3.15a-2

Find minimum required thickness for plate based on bending at tension interface:

Find tension bending line distance from edge of plate (AISC DG#1, 3.1.3):

$$x_{ten} := f_{dim} - \frac{0.8 \cdot OD_{post}}{2}$$

$x_{ten} = 1.35\ in$

Calculating minimum thickness based on tension:

$$t_{p\_ten\_req} := 2.11 \cdot \sqrt{\frac{T_u \cdot x_{ten}}{B_{plate} \cdot F_{y\_plate}}}$$

$t_{p\_ten\_req} = 0.331\ in$ *AISC DG#1 Eqn. 3.4.7a*

Controlling minimum required base plate thickness:

$$t_{p\_req} := \max\left(t_{p\_brng\_req}, t_{p\_ten\_req}\right)$$

$t_{p\_req} = 0.471\ in$

Check chosen plate thickness:

$$Plate\_Thick\_Check := \text{if } t_p \geq t_{p\_req} \; \Big\| \text{"Chosen plate thickness is adequate."} \; \text{else} \; \Big\| \text{"Need a thicker plate."}$$

$Plate\_Thick\_Check =$ "Chosen plate thickness is adequate."

Pipe to Plate Fillet Weld Connection Design

Given:

Minimum Fillet Weld Size: $w_{min} := \frac{1}{8}\ in$ *Min fillet weld size based on AISC Table J2-4*

Chosen fillet weld size $w := \frac{5}{16}\ in$

Weld material: $F_{EXX} := 70\ ksi$

Output:

Welded Connection to Base Plate Design:

Gross Length of Weld is Post Perimeter: $L_g := \pi \cdot OD_{post}$ $L_g = 9.032\ in$

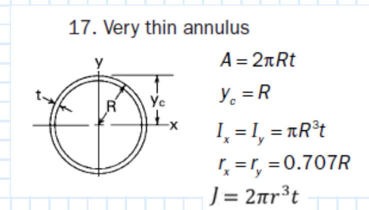
Effective Length of Weld: $L_w := L_g - 2 \cdot w$ $L_w = 8.407\ in$

Effective Throat Thickness: $t_e := min\left(w \cdot \sin(45\ deg), \frac{L_w}{4}\right)$ $t_e = 0.221\ in$ *AISC, Sect. J2, Pts. 2a & 2b*

Area of Weld: $A_w := L_w \cdot t_e$ $A_w = 1.858\ in^2$ *AISC, Sect. J2, Pts. 2a*

Moment of Inertia of circular fillet weld: $I_w := \pi \cdot \left(\frac{OD_{post}}{2}\right)^3 \cdot t_e$ $I_w = 2.062\ in^4$

Polar moment of Inertia of circular fillet weld: $J_w := 2\ \pi \cdot \left(\frac{OD_{post}}{2}\right)^3 \cdot t_e$ $J_w = 4.124\ in^4$



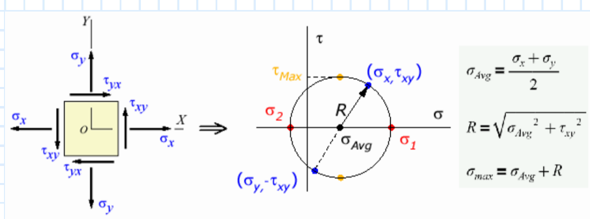
Determine design strength of weld:

Nominal strength of weld metal: $F_w := \phi_{fw} \cdot 0.6 \cdot F_{EXX}$ $F_w = 31.5\ ksi$ *AISC, Tbl. J2.5*

Normal stress caused by bending moment: $\sigma_b := \frac{M_{post\_u} \cdot \left(\frac{OD_{post}}{2}\right)}{I_w}$ $\sigma_b = 29.278\ ksi$ $\sigma = \frac{M}{S} = \frac{M \cdot c}{I}$

Stress caused by shearing force: $\tau_v := \frac{V_{post\_u}}{A_w}$ $\tau_v = 0.377\ ksi$

Resultant stress in weld from loading: $\sigma_{max} := \frac{\sigma_b}{2} + \sqrt{\left(\frac{\sigma_b}{2}\right)^2 + \tau_v^{\ 2}}$ $\sigma_{max} = 29.283\ ksi$

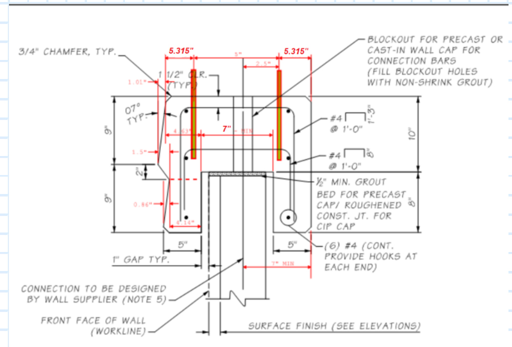


Check of weld thickness:

$Weld\_Design\_Check :=$ if $F_w \ge \sigma_{max}$
  "Chosen weld size is sufficient."
else
  "Need bigger fillet weld."

$Weld\_Design\_Check =$ "Chosen weld size is sufficient."

Anchor Bolt Connection Design



WALL CAP TYPICAL DETAIL

Given:

Number of anchor bolts resisting loads: $n_{ab} := 2$ *Only one side's bolts resist tension or shear.*

Bolts are specified as ASTM F1554 and Grade A36

Bolt diameter: $d_{ab} := \frac{5}{8}\ in$ *Plans*

Bolt area: $A_b := 0.307\ in^2$ *AISC Tbl. 7-18*

Bolt nominal yield stress strength: $F_{y\_bolt} := 36\ ksi$ *AISC Tbl. 2-3*

Bolt nominal ultimate tensile stress strength: $F_{u\_bolt} := 58\ ksi$

Bolt embedment: $h_{ef} := 5\ in$ *Plans*

Output:

Tension anchor bolt spacing: $s_1 := \frac{B_{plate} - 2 \cdot x_{bolt}}{n_{ab} - 1}$ $s_1 = 7\ in$

Bolt nominal tensile stress strength: $F_{nt} := 0.75 \cdot F_{u\_bolt}$ $F_{nt} = 43.5\ ksi$ *AISC Tbl. J3.2*

Bolt nominal shear stress strength: $F_{nv} := 0.40 \cdot F_{u\_bolt}$ $F_{nv} = 23.2\ ksi$ *AISC Tbl. J3.2, assuming threads within shear plane*

Ultimate tension load on one anchor bolt: $T_{u\_ab} := \frac{T_u}{n_{ab}}$ $T_{u\_ab} = 3.285\ kip$

Required shear stress on one bolt: $f_v := \frac{V_{post\_u}}{n_{ab} \cdot A_b}$ $f_v = 1.14\ ksi$

Bolt modified nominal tensile stress strength, modified for effects of shear stress: $F_{nt}' := min\left(F_{nt}, 1.3 \cdot F_{nt} - \frac{F_{nt}}{\phi_{ab} \cdot F_{nv}} \cdot f_v\right)$ $F_{nt}' = 43.5\ ksi$ *AISC Eqn. J3-3a*

Bolt factored tensile resistance: $\phi R_{n\_bolt} := \phi_{ab} \cdot F_{nt}' \cdot A_b$ $\phi R_{n\_bolt} = 10.016\ kip$ *AISC Eqn. J3-2*

Check of bolt tensile stress:

$Bolt\_Tensile\_Check :=$ if $\phi R_{n\_bolt} \ge T_{u\_ab}$
  "Bolt is satisfactory."
else
  "Bolt is no good."

$Bolt\_Tensile\_Check =$ "Bolt is satisfactory."

Continuing Anchor Bolt Connection Design per ACI 318

Outside diameter of anchor: $d_a := d_{ab}$ $d_a = 0.625\ in$

Critical edge distance for adhesive anchors: $c_{ac} := 2\ h_{ef}$ $c_{ac} = 10\ in$ *ACI 17.7.6*

Steel strength of anchor in tension (ACI 17.4.1)

Steel tension strength of anchor is confirmed above; so, no check here is necessary.

Concrete breakout strength of anchor in tension (ACI 17.4.2)

Check bolt group action for tension concrete breakout: *ACI 17.2.1.1*

$Group\_Tension\_Breakout\_Check :=$ if $s_1 \le 3 \cdot h_{ef}$
  "Bolts act in group."
else
  "Bolts act singly."

$Group\_Tension\_Breakout\_Check =$ "Bolts act in group."

Theoretical projected influence area of a single bolt far from an edge: $A_{Nco} := 9 \cdot h_{ef}^2$ $A_{Nco} = 225\ in^2$ *ACI Eqn. 17.4.2.1c*

Actual projected influence area for bolt(s): $A_{Nc} := min\left(\left(c_{a1} + 1.5 \cdot h_{ef}\right) \cdot \left(1.5 \cdot h_{ef} + min\left(s_1, 3 \cdot h_{ef}\right) + min\left(1.5 \cdot h_{ef}, c_{a2}\right)\right), n_{ab} \cdot A_{Nco}\right)$ $A_{Nc} = 243.485\ in^2$ *ACI Fig. R17.4.2.1*

Concrete tension breakout strength coefficient: $k_c := 17$ *Value of 17 for post-installed anchors, per ACI 17.4.2.2*

Basic concrete tension breakout strength for single anchor: $N_b := k_c \cdot 1.0 \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{h_{ef}}{in}\right)^{1.5} \cdot lbf$ $N_b = 12.021$ ***kip*** *ACI Eqn. 17.4.2.2a*

Factor for eccentrically loaded anchor bolts: $\Psi_{ec\_N} := 1.0$ *Anchor bolts are not loaded eccentrically.* *ACI 17.4.2.4*

Factor for anchor bolts near an edge: $\Psi_{ed\_N} := min\left(1.0, 0.7 + 0.3 \cdot \frac{c_{a1}}{1.5 \cdot h_{ef}}\right)$ $\Psi_{ed\_N} = 0.913$ *ACI Eqn. 17.4.2.5b*

Factor for anchor bolts in un-cracked concrete: $\Psi_{c\_N} := 1.4$ *Wall caps are not under load, and per the wall cap design, service moment from post does not cause cracking.* *ACI 17.4.2.6*

Factor for anchor bolts in un-cracked concrete near an edge without supplementary reinforcement: $\Psi_{cp\_N} := min\left(1.0, \max\left(\frac{c_{a1}}{c_{ac}}, \frac{1.5 \cdot h_{ef}}{c_{ac}}\right)\right)$ $\Psi_{cp\_N} = 0.75$ *ACI Eqn. 17.4.2.7b*

Nominal concrete tension breakout strength: $\phi N_{cbg} := \phi_{adh} \cdot \frac{A_{Nc}}{A_{Nco}} \cdot \Psi_{ec\_N} \cdot \Psi_{ed\_N} \cdot \Psi_{c\_N} \cdot \Psi_{cp\_N} \cdot N_b$ $\phi N_{cbg} = 8.102$ ***kip*** *ACI Eqn. 17.4.2.1b*

Check of concrete tension breakout failure:

$Concrete\_Tension\_Breakout\_Check :=$ if $\phi N_{cbg} \geq n_{ab} \cdot T_{u\_ab}$
  "Bolt is satisfactory."
else
  "Bolt is no good."

$Concrete\_Tension\_Breakout\_Check =$ "Bolt is satisfactory."

Pullout strength cast-in, post-installed expansion, or undercut anchor in tension (ACI 17.4.3)

Proposed anchors are post-installed adhesive, not headed studs or bolts, expansion anchors, or undercut anchors; so, no check is required.

Concrete side-face blowout strength of headed anchor in tension (ACI 17.4.4)

Proposed anchors are post-installed adhesive, not headed studs or bolts; so, no check is required.

Bond strength of adhesive anchor in tension (ACI 17.4.5)

Minimum bond stress for HY 200 Epoxy per HILTI ESR-3187:

$$\tau_{uncr\_HY\_200} := 0.65 \cdot \left(\frac{f'_c}{2500\ psi}\right)^{0.1} \cdot 2220\ psi = 1512.441\ psi$$

Per HILTI ESR-3187 Table 14, basic un-cracked bond strength is 2,220 psi; this value is factored by a straight 0.65 for either wet or dry installation conditions and by a small boost from concrete strength higher than 2,500 psi

Minimum bond stress for HIT-RE 500 Epoxy per HILTI ESR-3814:

$$\tau_{uncr\_HIT\_RE\_500} := 0.65 \cdot \left(\frac{f'_c}{2500\ psi}\right)^{0.15} \cdot 2210\ psi = 1541.429\ psi$$

Per HILTI ESR-3814 Table 12, basic un-cracked bond strength is 2,210 psi. This value is based on diamond coring and roughening afterwards; it is lower than being hammer-drilled with carbide bit. The socket must be roughened if coring with a diamond bit; this should be written on the plans. Factors are a straight 0.65 reduction factor independent of wet or dry concrete conditions during installation and a small boost for using concrete higher than 2,500 psi. The smaller factor for cracked concrete is used since no supplementary rebar is being provided; this also matches with reduction factor below.

Minimum bond stress strength: $\tau_{uncr} := min\left(\tau_{uncr\_HY\_200}, \tau_{uncr\_HIT\_RE\_500}\right)$ $\tau_{uncr} = 1512.441$ ***psi***

Distance to edge of project influence area: $c_{Na} := 10 \cdot d_a \cdot \sqrt{\frac{\tau_{uncr}}{1100\ psi}}$ $c_{Na} = 7.329$ ***in*** *ACI Eqn. 17.4.5.1d*

Check if anchor bolts act in group for bond failure: *ACI 17.2.1.1*

$Group\_Bond\_Failure\_Check :=$ if $s_1 \leq 2\ c_{Na}$
  "Bolts act in group."
else
  "Bolts act singly."

$Group\_Bond\_Failure\_Check =$ "Bolts act in group."

Theoretical projected influence area of a single bolt far from an edge: $A_{Nao} := (2\ c_{Na})^2$ $A_{Nao} = 214.835\ in^2$ *ACI Eqn. 17.4.5.1c*

Actual projected influence area for bolt(s): $A_{Na} := min\left(\left(c_{Na} + min\left(s_1, 2 \cdot c_{Na}\right) + min\left(c_{Na}, c_{a2}\right)\right) \cdot \left(c_{a1} + c_{Na}\right), n_{ab} \cdot A_{Nao}\right)$ $A_{Na} = 238.062\ in^2$ *ACI Fig. R17.4.5.1*

Basic bond strength of adhesive anchor: $N_{ba} := \tau_{uncr} \cdot \pi \cdot d_a \cdot h_{ef}$ $N_{ba} = 14.848\ kip$ *ACI Eqn. 17.4.5.2*

Concrete is not light weight; so, lambda-a is set to 1.0; per ACI 17.4.5.2, un-cracked bond stress may be used.

Factor for eccentrically loaded anchor bolts: $\Psi_{ec\_Na} := 1.0$ *Anchor bolts are not loaded eccentrically.* *ACI 17.4.5.3*

Factor for anchor bolts near an edge: $\Psi_{ed\_Na} := min\left(1.0, 0.7 + 0.3 \cdot \frac{c_{a1}}{c_{Na}}\right)$ $\Psi_{ed\_Na} = 0.918$ *ACI Eqn. 17.4.5.4b*

Factor for anchor bolts in un-cracked concrete near an edge without supplementary reinforcement: $\Psi_{cp\_Na} := min\left(1.0, \max\left(\frac{c_{a1}}{c_{ac}}, \frac{c_{Na}}{c_{ac}}\right)\right)$ $\Psi_{cp\_Na} = 0.733$ *ACI Eqn. 17.4.5.5b*

Wall caps are not under load, and per the wall cap design, service moment from post does not cause cracking.

Nominal bond strength of the adhesive anchor(s): $\phi N_{ag} := \phi_{adh} \cdot \frac{A_{Na}}{A_{Nao}} \cdot \Psi_{ec\_Na} \cdot \Psi_{ed\_Na} \cdot \Psi_{cp\_Na} \cdot N_{ba}$ $\phi N_{ag} = 7.192\ kip$ *ACI Eqn. 17.4.5.1b*

Check of bolt bond stress failure:

```
Bond_Stress_Check := if ϕN_ag ≥ n_ab • T_u_ab
                        ‖ "Bolt is satisfactory."
                     else
                        ‖ "Bolt is no good."
```

Bond_Stress_Check = "Bolt is satisfactory."

Steel strength of anchor in shear (17.5.1)

Steel shear strength of anchor is confirmed above; so, no check here is necessary.

Concrete breakout strength of anchor in shear (17.5.2)

Check bolt group action for shear concrete breakout: *ACI 17.2.1.1*

```
Group_Shear_Breakout_Check := if s_1 ≤ 3 • c_a1
                                 ‖ "Bolts act in group."
                              else
                                 ‖ "Bolts act singly."
```

Group_Shear_Breakout_Check = "Bolts act in group."

Theoretical projected influence area of a single bolt far from an edge: $A_{Vco} := 4.5 \cdot {c_{a1}}^2$ $A_{Vco} = 127.122\ in^2$ *ACI Eqn. 17.5.2.1c*

Actual projected influence area for bolt(s): $A_{Vc} := min\left(1.5 \cdot c_{a1} \cdot \left(1.5 \cdot c_{a1} + min\left(s_1, 3 \cdot c_{a1}\right) + min\left(1.5 \cdot c_{a1}, c_{a2}\right)\right), n_{ab} \cdot A_{Vco}\right)$ $A_{Vc} = 155.245\ in^2$ *ACI Fig. R17.5.2.1b*

Load bearing length: $l_e := h_{ef}$ $l_e = 5\ in$ *ACI 17.5.2.2*

Basic concrete breakout strength in shear for single anchor:

$$V_b := min\left(\left(7\left(\frac{l_e}{d_a}\right)^{0.2} \cdot \sqrt{\frac{d_a}{in}}\right) \cdot 1.0 \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{c_{a1}}{in}\right)^{1.5}, 9 \cdot 1.0 \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{c_{a1}}{in}\right)^{1.5}\right) \cdot lbf \qquad V_b = 6.5\ kip$$

Concrete is not light weight; so, lambda-a is set to 1.0. *ACI Eqns. 17.5.2.2a & 17.5.2.2b*

Factor for eccentrically loaded anchor bolts: $\Psi_{ec\_V} := 1.0$ *Anchor bolts are not loaded eccentrically.* *ACI 17.5.2.5*

Factor for anchor bolts near an edge: $\Psi_{ed\_V} := min\left(1.0, 0.7 + 0.3 \cdot \frac{c_{a2}}{1.5 \cdot c_{a1}}\right)$ $\Psi_{ed\_V} = 0.869$ *ACI Eqns. 17.5.2.6a & 17.5.2.6b*

Factor for anchor bolts in un-cracked concrete: $\Psi_{c\_V} := 1.4$ *Wall caps are not under load, and per the wall cap design, service moment from post does not cause cracking.* *ACI 17.5.2.7*

Factor for small embedment $\Psi_{h\_V} := min\left(1.0, \sqrt{\frac{1.5 \cdot c_{a1}}{h_{ef}}}\right)$ $\Psi_{h\_V} = 1$ *ACI Eqn. 17.5.2.8*

Nominal concrete shear breakout strength: $\phi V_{cbg} := \phi_{adh} \cdot \frac{A_{Vc}}{A_{Vco}} \cdot \Psi_{ec\_V} \cdot \Psi_{ed\_V} \cdot \Psi_{c\_V} \cdot \Psi_{h\_V} \cdot V_b$ $\phi V_{cbg} = 6.28\ \textbf{kip}$ *ACI Eqn. 17.5.2.1b*

Check of concrete shear breakout failure: *Concrete_Shear_Breakout_Check* := if $\phi V_{cbg} \geq V_{post\_u}$
  "Bolt is satisfactory."
else
  "Bolt is no good."

Concrete_Shear_Breakout_Check = "Bolt is satisfactory."

Concrete pryout strength of anchor in shear (17.5.3)

Basic concrete pryout strength of a single anchor in shear: $\phi N_{cpg} := min\left(\phi N_{ag}, \phi N_{cbg}\right)$ $\phi N_{cpg} = 7.192\ \textbf{kip}$ *ACI 17.5.3.1*

Concrete pryout strength in shear coefficient: k_{cp} := if $h_{ef} < 2.5\ \textbf{in}$
  1.0
else
  2.0

$k_{cp} = 2$ *ACI 17.5.3.1*

Nominal concrete pryout strength of anchor(s) in shear: $\phi V_{cpg} := k_{cp} \cdot \phi N_{cpg}$ $\phi V_{cpg} = 14.384\ \textbf{kip}$ *ACI Eqn. 17.5.3.1b*

Check of concrete pryout strength in shear: *Concrete_Shear_Pryout_Check* := if $\phi V_{cpg} \geq V_{post\_u}$
  "Bolt is satisfactory."
else
  "Bolt is no good."

Concrete_Shear_Pryout_Check = "Bolt is satisfactory."

Vertical Interior Post and Horizontal Rail Design

Given:

Post Height: $H_{post} := 42\ in$ *Plans*

Post and Rail Yield Strength: $F_y := 50\ ksi$ *ASTM F1043*

Post and Rail Modulus of Elasticity: $E_s := 29000\ ksi$

Post and Rail Ultimate Strength: $F_u := 58\ ksi$

Properties for ASTM F1043 IC 1-7/8" Pipe for Interior Posts

Post OD: $OD_{post} := 1.900\ in$

Post Thickness: $t_{post} := 0.12\ in$

Properties for ASTM F1043 IC 1-5/8" Pipe for Rails

Rail OD: $OD_{rail} := 1.66\ in$

Rail Thickness: $t_{rail} := 0.111\ in$

Design Point Live Load $P_{LL} := 200\ lbf$ *AASHTO 13.8.2*

Design Uniform Live Load $w_{LL} := 0\ plf$ *AASHTO 13.8.2*

Post spacing: $L_{spc} := 8\ ft$ *Plans*

Weight of chain link fence: $f_{clf} := 0.48\ psf$

Design wind load from chain link fence: $f_{wind} := 15\ psf$ *AASHTO 13.8.2*

Load Factors (AASHTO Tbl. 3.4.1-1):

PL Load Factor: $\gamma_{PL} := 1.75$

DC Load Factor: $\gamma_{DL} := 1.25$

WS Load Factor: $\gamma_{WS} := 1.00$

Resistance Factors:

Steel Flexure (AASHTO 6.5.4.2): $\phi_f := 1.00$

Steel Shear (AASHTO 6.5.4.2): $\phi_v := 1.00$

Tension, Yielding in Gross Section: $\phi_y := 0.95$

Bending (AISC F1): $\phi_b := 0.90$

Shear (AISC G1): $\phi_{v\_AISC} := 0.90$

Bearing (AISC DG#1): $\phi_{brg} := 0.60$

Fillet Weld (AISC Tbl. J2.5): $\phi_{fw} := 0.75$

Bolts (AISC J3.6 & J3.7): $\phi_{ab} := 0.75$

Adhesive Anchor Bolts (ACI 17.3.3, Condition B, Category 1): $\phi_{adh} := 0.65$

Steel weight density: $\gamma_{steel} := 490\ pcf$

ASTM F1043 Group IC Electric Resistant Welded 50,000 psi yield steel pipe

Trade	Decimal O.D.		Pipe wall				Section			Min. Yield			Max Bending	Calculated Load (lbs)		
Reference	Equivalent		Thickness		Weight		Modulus		x	Strength		=	Moment	10' Free	Cantilever	
O.D.	inches	(mm)	inches	(mm)	lb./ft.	(kg/m)	inches³	(mm³)	x	psi	(Mpa)	=	Lb.In.	Supported	4'	6'
1 5/8"	1.660	42.16	0.111	2.82	1.84	2.74	0.1962	4.98	x	50000	345	=	9810	327	204	136
1 7/8"	1.900	48.26	0.120	3.05	2.28	3.39	0.2810	7.14	x	50000	345	=	14050	468	293	195
2 3/8"	2.375	60.33	0.130	3.30	3.12	4.64	0.4881	12.40	x	50000	345	=	24405	814	508	339
2 7/8"	2.875	73.03	0.160	4.06	4.64	6.91	0.8778	22.30	x	50000	345	=	43890	1463	914	610
3 1/2"	3.500	88.90	0.160	4.06	5.71	8.50	1.3408	34.06	x	50000	345	=	67042	2235	1397	931
4"	4.000	101.60	0.160	4.06	6.57	9.78	1.7820	45.26	x	50000	345	=	89098	2970	1856	1237
4 1/2"	4.500	114.30	0.160	4.14	7.42	11.04	2.2859	57.99	X	50000	345	=	114295	3810	5486	1587

Output:

Post Section Properties:

Post inside diameter: $ID_{post} := OD_{post} - 2 \cdot t_{post}$ $ID_{post} = 1.66\ in$

Post Area: $A_{post} := 0.785398 \cdot \left(OD_{post}^2 - ID_{post}^2\right)$ $A_{post} = 0.671\ in^2$

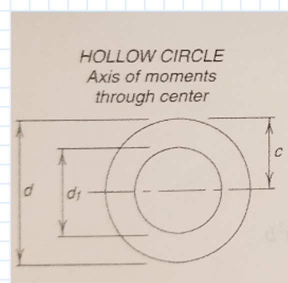
Post Unit Weight: $w_{post} := \gamma_{steel} \cdot A_{post}$ $w_{post} = 2.283\ plf$

Post centroid: $c_{post} := 0.5 \cdot OD_{post}$ $c_{post} = 0.95\ in$

Post Moment of Inertial: $I_{post} := 0.049087 \cdot \left(OD_{post}^4 - ID_{post}^4\right)$ $I_{post} = 0.267\ in^4$

Post Section Modulus: $S_{post} := \dfrac{I_{post}}{c_{post}}$ $S_{post} = 0.281\ in^3$

Post Plastic Section Modulus: $Z_{post} := \dfrac{OD_{post}^3 - ID_{post}^3}{6}$ $Z_{post} = 0.381\ in^3$



$$A = \frac{\pi(d^2 - d_1^2)}{4} = .785398\,(d^2 - d_1^2)$$

$$c = \frac{d}{2}$$

$$I = \frac{\pi(d^4 - d_1^4)}{64} = .049087\,(d^4 - d_1^4)$$

$$S = \frac{\pi(d^4 - d_1^4)}{32\,d} = .098175\,\frac{d^4 - d_1^4}{d}$$

$$r = \frac{\sqrt{d^2 + d_1^2}}{4}$$

$$Z = \frac{d^3}{6} - \frac{d_1^3}{6}$$

Rail Section Properties:

Rail inside diameter: $ID_{rail} := OD_{rail} - 2 \cdot t_{rail}$ $ID_{rail} = 1.438\ in$

Rail Area: $A_{rail} := 0.785398 \cdot \left(OD_{rail}^2 - ID_{rail}^2\right)$ $A_{rail} = 0.54\ in^2$

Rail Unit Weight: $w_{rail} := \gamma_{steel} \cdot A_{rail}$ $w_{rail} = 1.838\ plf$

Rail centroid: $c_{rail} := 0.5 \cdot OD_{rail}$ $c_{rail} = 0.83\ in$

Rail Moment of Inertial: $I_{rail} := 0.049087 \cdot \left(OD_{rail}^4 - ID_{rail}^4\right)$ $I_{rail} = 0.163\ in^4$

Rail Section Modulus: $S_{rail} := \dfrac{I_{rail}}{c_{rail}}$ $S_{rail} = 0.196\ in^3$

Rail Plastic Section Modulus: $Z_{rail} := \dfrac{OD_{rail}^3 - ID_{rail}^3}{6}$ $Z_{rail} = 0.267\ in^3$

Post concentrated live load applied at top rail: $P_{post\_LL} := P_{LL} + w_{LL} \cdot L_{spc} = 0.2\ kip$ $\quad P_{post\_LL} = 0.2\ kip$ *AASHTO Eqn. 13.8.2-1*

Post moment loading from live load: $M_{post\_LL} := P_{post\_LL} \cdot H_{post} = 8400\ lbf \cdot in$ $\quad M_{post\_LL} = 8400\ lbf \cdot in$ *Post treated as cantilevered beam*

Post shear from live load: $V_{post\_LL} := P_{post\_LL}$ $\quad V_{post\_LL} = 0.2\ kip$

Rail moment from live load applied: $M_{rail\_LL} := \sqrt{2} \cdot \dfrac{w_{LL} \cdot L_{spc}^{\ 2}}{8} + \dfrac{P_{LL} \cdot L_{spc}}{4}$ $\quad M_{rail\_LL} = 4800\ lbf \cdot in$

Rail treated as simply supported beam with vertical and horizontal live loads combined into resultant direction.

Rail moment from dead load: $M_{rail\_DL} := \dfrac{w_{rail} \cdot L_{spc}^{\ 2}}{8} + \dfrac{f_{clf} \cdot \dfrac{H_{post}}{2} \cdot L_{spc}^{\ 2}}{8}$ $\quad M_{rail\_DL} = 257.093\ lbf \cdot in$

Rail shear from live load: $V_{rail\_LL} := \sqrt{2} \cdot \dfrac{w_{LL} \cdot L_{spc}}{2} + \dfrac{P_{LL}}{2}$ $\quad V_{rail\_LL} = 0.1\ kip$

Rail shear from dead load: $V_{rail\_DL} := \dfrac{w_{rail} \cdot L_{spc}}{2} + \dfrac{f_{clf} \cdot \dfrac{H_{post}}{2} \cdot L_{spc}}{2}$ $\quad V_{rail\_DL} = 0.011\ kip$

Factored Shear Load on Post: $V_{post\_u} := \gamma_{PL} \cdot V_{post\_LL}$ $\quad V_{post\_u} = 0.35\ kip$

Factored Moment Load on Post: $M_{post\_u} := \gamma_{PL} \cdot M_{post\_LL}$ $\quad M_{post\_u} = 14700\ lbf \cdot in$

AASHTO load factors used instead of ASCE load factors found in AISC and ACI. This is acceptable as it is more conservative.

Factored Shear Load on Rail: $V_{rail\_u} := \gamma_{PL} \cdot V_{rail\_LL} + \gamma_{DL} \cdot V_{rail\_DL}$ $\quad V_{rail\_u} = 0.188\ kip$

Factored Moment Load on Rail: $M_{rail\_u} := \gamma_{PL} \cdot M_{rail\_LL} + \gamma_{DL} \cdot M_{rail\_DL}$ $\quad M_{rail\_u} = 8721.366\ lbf \cdot in$

Vertical dead load was combined directly with live load resultant since it was so small compared to the live load.

Post Analysis:

Following AASHTO 6.12.1.2.3c for Shear Design:

Gross Area: $A_g := A_{post}$ $\quad A_g = 0.671\ in^2$

Distance from Max to 0 Shear: $L_v := H_{post}$ $\quad L_v = 42\ in$

Critical Strength for Shear:

$$F_{cr} := min\left(0.58 \cdot F_y, \max\left(\frac{1.6 \cdot E_s}{\sqrt{\dfrac{L_v}{OD_{post}}}\left(\dfrac{OD_{post}}{t_{post}}\right)^{\frac{5}{4}}}, \frac{0.78 \cdot E_s}{\left(\dfrac{OD_{post}}{t_{post}}\right)^{\frac{3}{2}}}\right)\right)$$

$F_{cr} = 29\ ksi$ *AASHTO Eqns. 6.12.1.2.3c-2 & 6.12.1.2.3c-3*

Factored nominal shear resistance: $\phi V_n := \phi_v \cdot 0.5\ F_{cr} \cdot A_g$ $\quad \phi V_n = 9.73\ kip$ *AASHTO Eqn. 6.12.1.2.3c-1*

Post Shear Check: $\dfrac{\phi V_n}{V_{post\_u}} = 27.8$

$Post\_Shear\_Check :=$ if $\dfrac{\phi V_n}{V_{post\_u}} \ge 1.0$
 "Post shear strength is satisfactory."
else
 "Post is not satisfactory."

$Post\_Shear\_Check =$ "Post shear strength is satisfactory."

Following AASHTO 6.12.2.2.3 for Flexure Design:

Check of Noncompact Section:

$Check\_Compact :=$ if $\dfrac{OD_{post}}{t_{post}} \le \dfrac{0.07 \cdot E_s}{F_y}$
 "Section is compact. Local buckling does not apply."
else
 "Section is not compact. Check wall slenderness."

Per AASHTO 6.12.2.2.3, as long D/t does not exceed 0.07E/Fy, plastic modulus and equation 6.12.2.2.3-1 may be used.

$Check\_Compact =$ "Section is compact. Local buckling does not apply."

Factored Nominal Moment Resistance: $\phi M_n := \phi_f \cdot F_y \cdot Z_{post}$ $\quad \phi M_n = 19.039\ kip \cdot in$ *AASHTO Eqn. 6.12.2.2.3-1*

Post Flexural Check:

$$\frac{\phi M_n}{M_{post\_u}} = 1.295$$

$$Post\_Flex\_Check := \text{if } \frac{\phi M_n}{M_{post\_u}} \ge 1.0 \;\; \| \text{"Post flexural strength is satisfactory."} \;\; \text{else} \;\; \| \text{"Post is not satisfactory."}$$

$Post\_Flex\_Check =$ "Post flexural strength is satisfactory."

Rail Analysis:

Following AASHTO 6.12.1.2.3c for Shear Design:

Gross Area: $A_g := A_{rail}$ $\qquad A_g = 0.54\ in^2$

Distance from Max to 0 Shear: $L_v := \frac{L_{spc}}{2}$ $\qquad L_v = 48\ in$

Critical Strength for Shear:

$$F_{cr} := min\left(0.58 \cdot F_y, \max\left(\frac{1.6 \cdot E_s}{\sqrt{\frac{L_v}{OD_{rail}}}\left(\frac{OD_{rail}}{t_{rail}}\right)^{\frac{5}{4}}}, \frac{0.78 \cdot E_s}{\left(\frac{OD_{rail}}{t_{rail}}\right)^{\frac{3}{2}}}\right)\right)$$

$F_{cr} = 29\ ksi$ AASHTO Eqns. 6.12.1.2.3c-2 & 6.12.1.2.3c-3

Factored Nominal Shear Resistance: $\phi V_n := \phi_v \cdot 0.5\ F_{cr} \cdot A_g$ $\qquad \phi V_n = 7.832\ kip$ AASHTO Eqn. 6.12.1.2.3c-1

Rail Shear Check:

$$\frac{\phi V_n}{V_{rail\_u}} = 41.575$$

$$Rail\_Shear\_Check := \text{if } \frac{\phi V_n}{V_{rail\_u}} \ge 1.0 \;\; \| \text{"Rail shear strength is satisfactory."} \;\; \text{else} \;\; \| \text{"Rail is not satisfactory."}$$

$Rail\_Shear\_Check =$ "Rail shear strength is satisfactory."

Following AASHTO 6.12.2.2.3 for Flexure Design:

Check of Noncompact Section:

$$Check\_Compact := \text{if } \frac{OD_{rail}}{t_{rail}} \le \frac{0.07 \cdot E_s}{F_y} \;\; \| \text{"Section is compact. Local buckling does not apply."} \;\; \text{else} \;\; \| \text{"Section is not compact. Check wall slenderness."}$$

Per AASHTO 6.12.2.2.3, as long D/t does not exceed 0.07E/Fy, plastic modulus and equation 6.12.2.2.3-1 may be used.

$Check\_Compact =$ "Section is compact. Local buckling does not apply."

Factored Nominal Moment Resistance: $\phi M_n := \phi_f \cdot F_y \cdot Z_{rail}$ $\qquad \phi M_n = 13.339\ kip \cdot in$ AASHTO Eqn. 6.12.2.2.3-1

Post Flexural Check:

$$\frac{\phi M_n}{M_{rail\_u}} = 1.53$$

$$Rail\_Flex\_Check := \text{if } \frac{\phi M_n}{M_{rail\_u}} \ge 1.0 \;\; \| \text{"Rail flexural strength is satisfactory."} \;\; \text{else} \;\; \| \text{"Rail is not satisfactory."}$$

$Rail\_Flex\_Check =$ "Rail flexural strength is satisfactory."

Confirming that Wind Loading Doesn't Control:

Per last paragraph of AASHTO 13.8.2, the wind load on the chain link fence is not applied simultaneously with the live load.

Uniform wind load on post: $w_{post\_wind} := f_{wind} \cdot L_{spc}$ $w_{post\_wind} = 120\ plf$

Design moment from wind on post: $M_{post\_wind\_u} := \gamma_{WS} \cdot \dfrac{w_{post\_wind} \cdot H_{post}^{2}}{2}$ $M_{post\_wind\_u} = 8820\ lbf \cdot in$

$M_{post\_u} = 14700\ lbf \cdot in$ <- LL controls

Design shear from wind on post: $V_{post\_wind\_u} := \gamma_{WS} \cdot w_{post\_wind} \cdot H_{post}$ $V_{post\_wind\_u} = 0.42\ kip$

$V_{post\_u} = 0.35\ kip$ <- LL controls

Uniform wind on rail: $w_{rail\_wind} := f_{wind} \cdot \dfrac{H_{post}}{2}$ $w_{rail\_wind} = 26.25\ plf$

Design moment from wind on rail: $M_{rail\_wind\_u} := \gamma_{WS} \cdot \dfrac{w_{rail\_wind} \cdot L_{spc}^{2}}{8}$ $M_{rail\_wind\_u} = 2520\ lbf \cdot in$

$M_{rail\_u} = 8721.366\ lbf \cdot in$ <- LL controls

Design shear from wind on rail: $V_{rail\_wind\_u} := \gamma_{WS} \cdot w_{rail\_wind} \cdot \dfrac{L_{spc}}{2}$ $V_{rail\_wind\_u} = 0.105\ kip$

$V_{rail\_u} = 0.188\ kip$ <- LL controls

Base Plate Design - Line Post w/ Axial Compression

Given: *Plans*

Cap width: $W_{cap} := 15.63\ in$

Distance from post to end of cap: $L_{end} := 96\ in$

Plate thickness: $t_p := 0.5\ in$

Plate length (perpendicular to fence): $N_{plate} := 8\ in$

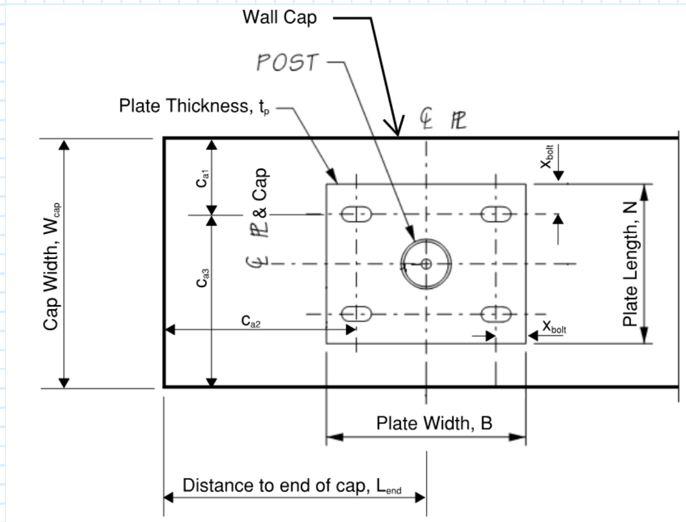
Plate width (parallel to fence): $B_{plate} := 10\ in$

Compressive Strength of Concrete: $f'_c := 4\ ksi$

Side clearance to anchor bolts: $x_{bolt} := 1.5\ in$

Base plate steel yield strength: $F_{y\_plate} := 36\ ksi$

Number of rails: $n_{rail} := 2$



Output:

Plate Area: $A_{plate} := N_{plate} \cdot B_{plate}$ $\quad A_{plate} = 80\ in^2$

Distance from bolt to near face of cap: $c_{a1} := \frac{1}{2}\left(W_{cap} - N_{plate}\right) + x_{bolt}$ $\quad c_{a1} = 5.315\ in$

Distance from outside bolt to end of cap: $c_{a2} := L_{end} - \frac{B_{plate}}{2} + x_{bolt}$ $\quad c_{a2} = 92.5\ in$

Distance from bolt to far face of cap: $c_{a3} := W_{cap} - c_{a1}$ $\quad c_{a3} = 10.315\ in$

Bearing Area taken to Be Same as Plate Area: $A_{bearing} := A_{plate}$ $\quad A_{bearing} = 80\ in^2$ *Conservatively setting bearing area to the same as the plate.*

Max allowed bearing pressure: $f_{pu\_max} := \phi_{brg} \cdot min\left(0.85 \cdot f'_c \cdot \sqrt{\frac{A_{bearing}}{A_{plate}}}, 1.7 \cdot f'_c\right)$ $\quad f_{pu\_max} = 2.04\ ksi$ *ACI Tbl. 14.5.6.1*

Max allowed bearing pressure line: $q_{max} := f_{pu\_max} \cdot B_{plate}$ $\quad q_{max} = \left(2.448 \cdot 10^5\right) \frac{lbf}{ft}$

Post dead load on plate $P_{post\_DL} := w_{post} \cdot H_{post}$ $\quad P_{post\_DL} = 0.008\ kip$

Rail dead load on plat: $P_{rail\_DL} := n_{rail} \cdot 2 \cdot V_{rail\_DL}$ $\quad P_{rail\_DL} = 0.043\ kip$

Factored vertical load on plate: $P_u := \gamma_{DL} \cdot \left(P_{post\_DL} + P_{rail\_DL}\right)$ $\quad P_u = 0.064\ kip$

Minimum length of area of bearing: $Y_{min} := \frac{P_u}{q_{max}}$ $\quad Y_{min} = 0.003\ in$ *AISC DG#1 Eqn. 3.3.3*

Critical eccentricity distance: $e_{crit} := \frac{N_{plate}}{2} - \frac{Y_{min}}{2}$ $\quad e_{crit} = 3.998\ in$ *AISC DG#1 Eqn. 3.3.7*

Eccentricity of loading: $e_{loading} := \frac{M_{post\_u}}{P_u}$ $\quad e_{loading} = 231.31\ in$ *AISC DG#1 Eqn. 3.3.6*

Small moment check:

Small_Moment_Check := if $e_{loading} \le e_{crit}$
  "Moment is small, no need for anchor bolts."
else
  "Moment is large, need anchor bolts."

Small_Moment_Check = "Moment is large, need anchor bolts."

Distance from bolt to center of post: $f_{dim} := \frac{N_{plate}}{2} - x_{bolt}$ $\quad f_{dim} = 2.5\ in$

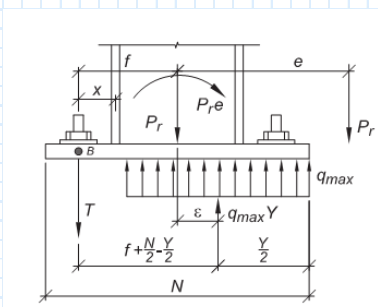


Fig. 3.4.1. Base plate with large moment.

Plate dimension check

$$Plate\_Dim\_Check := \text{if } \left(f_{dim} + \frac{N_{plate}}{2}\right)^2 \ge \frac{2 \cdot P_u \cdot (e_{loading} + f_{dim})}{q_{max}}$$
‖ "Plate dimensions are OK."
else
‖ "Plate needs to be longer and/or wider."

$Plate\_Dim\_Check =$ "Plate dimensions are OK."

Length of bearing area centered at the eccentricity of this loading:

$$Y_{loading} := \left(f_{dim} + \frac{N_{plate}}{2}\right) - \sqrt{\left(f_{dim} + \frac{N_{plate}}{2}\right)^2 - \frac{2 \cdot P_u \cdot (e_{loading} + f_{dim})}{q_{max}}}$$

$Y_{loading} = 0.113$ ***in***

AISC DG#1 Eqn. 3.4.3

Required tensile resistance in anchor rods: $T_u := q_{max} \cdot Y_{loading} - P_u$ $T_u = 2.242$ ***kip*** *AISC DG#1 Eqn. 3.4.2*

Find minimum required thickness for plate based on bending at bearing interface:

Find bearing bending line distance from edge of plate (AISC DG#1, 3.1.3):

$$m_{plate} := \frac{N_{plate} - 0.8\ OD_{post}}{2}$$

$m_{plate} = 3.24$ ***in***

Calculating minimum thickness based on bearing:

$$t_{p\_brng\_req} := \text{if } Y_{loading} \ge m_{plate}$$
$$\Vert\ 1.5 \cdot m_{plate} \cdot \sqrt{\frac{f_{pu\_max}}{F_{y\_plate}}}$$
else
$$\Vert\ 2.11 \cdot \sqrt{\frac{f_{pu\_max} \cdot Y_{loading} \cdot \left(m_{plate} - \frac{Y_{loading}}{2}\right)}{F_{y\_plate}}}$$

$t_{p\_brng\_req} = 0.301$ ***in***

AISC DG#1 Eqns. 3.3.14a-2 & 3.3.15a-2

Find minimum required thickness for plate based on bending at tension interface:

Find tension bending line distance from edge of plate (AISC DG#1, 3.1.3):

$$x_{ten} := f_{dim} - \frac{0.8 \cdot OD_{post}}{2}$$

$x_{ten} = 1.74$ ***in***

Calculating minimum thickness based on tension:

$$t_{p\_ten\_req} := 2.11 \cdot \sqrt{\frac{T_u \cdot x_{ten}}{B_{plate} \cdot F_{y\_plate}}}$$

$t_{p\_ten\_req} = 0.22$ ***in*** *AISC DG#1 Eqn. 3.4.7a*

Controlling minimum required base plate thickness: $t_{p\_req} := \max\left(t_{p\_brng\_req}, t_{p\_ten\_req}\right)$ $t_{p\_req} = 0.301$ ***in***

Check chosen plate thickness:

$$Plate\_Thick\_Check := \text{if } t_p \ge t_{p\_req}$$
‖ "Chosen plate thickness is adequate."
else
‖ "Need a thicker plate."

$Plate\_Thick\_Check =$ "Chosen plate thickness is adequate."

Pipe to Plate Fillet Weld Connection Design

Given:

Minimum Fillet Weld Size: $w_{min} := \frac{1}{8}\ in$ *Min fillet weld size based on AISC Table J2-4*

Chosen fillet weld size $w := \frac{5}{16}\ in$

Weld material: $F_{EXX} := 70\ ksi$

Output:

Welded Connection to Base Plate Design:

Gross Length of Weld is Post Perimeter: $L_g := \pi \cdot OD_{post}$ $L_g = 5.969\ in$

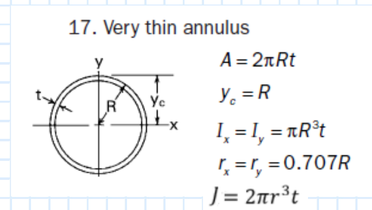
Effective Length of Weld: $L_w := L_g - 2 \cdot w$ $L_w = 5.344\ in$

Effective Throat Thickness: $t_e := min\left(w \cdot \sin(45\ deg), \frac{L_w}{4}\right)$ $t_e = 0.221\ in$ *AISC, Sect. J2, Pts. 2a & 2b*

Area of Weld: $A_w := L_w \cdot t_e$ $A_w = 1.181\ in^2$ *AISC, Sect. J2, Pts. 2a*

Moment of Inertia of circular fillet weld: $I_w := \pi \cdot \left(\frac{OD_{post}}{2}\right)^3 \cdot t_e$ $I_w = 0.595\ in^4$

Polar moment of Inertia of circular fillet weld: $J_w := 2\ \pi \cdot \left(\frac{OD_{post}}{2}\right)^3 \cdot t_e$ $J_w = 1.19\ in^4$



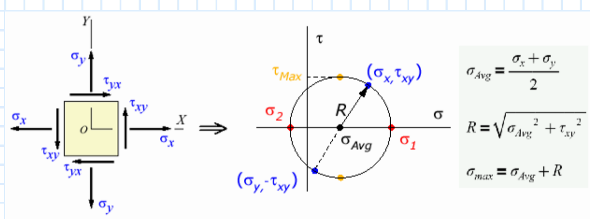
Determine design strength of weld:

Nominal strength of weld metal: $F_w := \phi_{fw} \cdot 0.6 \cdot F_{EXX}$ $F_w = 31.5\ ksi$ *AISC, Tbl. J2.5*

Normal stress caused by bending moment: $\sigma_b := \frac{M_{post\_u} \cdot \left(\frac{OD_{post}}{2}\right)}{I_w}$ $\sigma_b = 23.463\ ksi$ $\sigma = \frac{M}{S} = \frac{M \cdot c}{I}$

Stress caused by shearing force: $\tau_v := \frac{V_{post\_u}}{A_w}$ $\tau_v = 0.296\ ksi$

Resultant stress in weld from loading: $\sigma_{max} := \frac{\sigma_b}{2} + \sqrt{\left(\frac{\sigma_b}{2}\right)^2 + \tau_v^{\,2}}$ $\sigma_{max} = 23.467\ ksi$

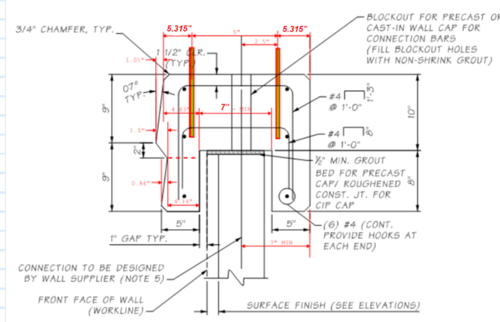


Check of weld thickness:

$Weld\_Design\_Check :=$ if $F_w \ge \sigma_{max}$
 "Chosen weld size is sufficient."
else
 "Need bigger fillet weld."

$Weld\_Design\_Check =$ "Chosen weld size is sufficient."

Anchor Bolt Connection Design



WALL CAP TYPICAL DETAIL

Given:

Number of anchor bolts resisting loads: $n_{ab} := 2$ *Only one side's bolts resist tension or shear.*

Bolts are specified as ASTM F1554 and Grade A36

Bolt diameter: $d_{ab} := \frac{5}{8}\ \mathbf{in}$ *Plans*

Bolt area: $A_b := 0.307\ \mathbf{in}^2$ *AISC Tbl. 7-18*

Bolt nominal yield stress strength: $F_{y\_bolt} := 36\ \mathbf{ksi}$ *AISC Tbl. 2-3*

Bolt nominal ultimate tensile stress strength: $F_{u\_bolt} := 58\ \mathbf{ksi}$

Bolt embedment: $h_{ef} := 5\ \mathbf{in}$ *Plans*

Output:

Tension anchor bolt spacing: $s_1 := \dfrac{B_{plate} - 2 \cdot x_{bolt}}{n_{ab} - 1}$ $s_1 = 7\ \mathbf{in}$

Bolt nominal tensile stress strength: $F_{nt} := 0.75 \cdot F_{u\_bolt}$ $F_{nt} = 43.5\ \mathbf{ksi}$ *AISC Tbl. J3.2*

Bolt nominal shear stress strength: $F_{nv} := 0.40 \cdot F_{u\_bolt}$ $F_{nv} = 23.2\ \mathbf{ksi}$ *AISC Tbl. J3.2, assuming threads within shear plane*

Ultimate tension load on one anchor bolt: $T_{u\_ab} := \dfrac{T_u}{n_{ab}}$ $T_{u\_ab} = 1.121\ \mathbf{kip}$

Required shear stress on one bolt: $f_v := \dfrac{V_{post\_u}}{n_{ab} \cdot A_b}$ $f_v = 0.57\ \mathbf{ksi}$

Bolt modified nominal tensile stress strength, modified for effects of shear stress: $F_{nt}' := min\left(F_{nt}, 1.3 \cdot F_{nt} - \dfrac{F_{nt}}{\phi_{ab} \cdot F_{nv}} \cdot f_v\right)$ $F_{nt}' = 43.5\ \mathbf{ksi}$ *AISC Eqn. J3-3a*

Bolt factored tensile resistance: $\phi R_{n\_bolt} := \phi_{ab} \cdot F_{nt}' \cdot A_b$ $\phi R_{n\_bolt} = 10.016\ \mathbf{kip}$ *AISC Eqn. J3-2*

Check of bolt tensile stress:

$$Bolt\_Tensile\_Check := \begin{array}{l} \text{if } \phi R_{n\_bolt} \ge T_{u\_ab} \\ \quad \| \text{“Bolt is satisfactory.”} \\ \text{else} \\ \quad \| \text{“Bolt is no good.”} \end{array}$$

$Bolt\_Tensile\_Check =$ “Bolt is satisfactory.”

Continuing Anchor Bolt Connection Design per ACI 318

Outside diameter of anchor: $d_a := d_{ab}$ $d_a = 0.625\ \mathbf{in}$

Critical edge distance for adhesive anchors: $c_{ac} := 2\ h_{ef}$ $c_{ac} = 10\ \mathbf{in}$ *ACI 17.7.6*

Steel strength of anchor in tension (ACI 17.4.1)

Steel tension strength of anchor is confirmed above; so, no check here is necessary.

Concrete breakout strength of anchor in tension (ACI 17.4.2)

Check bolt group action for tension concrete breakout:

$$Group\_Tension\_Breakout\_Check := \begin{array}{l} \text{if } s_1 \le 3 \cdot h_{ef} \\ \quad \| \text{“Bolts act in group.”} \\ \text{else} \\ \quad \| \text{“Bolts act singly.”} \end{array}$$

ACI 17.2.1.1

$Group\_Tension\_Breakout\_Check =$ “Bolts act in group.”

Theoretical projected influence area of a single bolt far from an edge: $A_{Nco} := 9 \cdot h_{ef}^2$ $A_{Nco} = 225\ \mathbf{in}^2$ *ACI Eqn. 17.4.2.1c*

Actual projected influence area for bolt(s): $A_{Nc} := min\left(\left(c_{a1} + 1.5 \cdot h_{ef}\right) \cdot \left(1.5 \cdot h_{ef} + min\left(s_1, 3 \cdot h_{ef}\right) + min\left(1.5 \cdot h_{ef}, c_{a2}\right)\right), n_{ab} \cdot A_{Nco}\right)$ $A_{Nc} = 281.93\ \mathbf{in}^2$ *ACI Fig. R17.4.2.1*

Concrete tension breakout strength coefficient: $k_c := 17$ *Value of 17 for post-installed anchors, per ACI 17.4.2.2*

Basic concrete tension breakout strength for single anchor: $N_b := k_c \cdot 1.0 \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{h_{ef}}{in}\right)^{1.5} \cdot lbf$ $N_b = 12.021$ ***kip*** *ACI Eqn. 17.4.2.2a*

Factor for eccentrically loaded anchor bolts: $\Psi_{ec\_N} := 1.0$ *Anchor bolts are not loaded eccentrically.* *ACI 17.4.2.4*

Factor for anchor bolts near an edge: $\Psi_{ed\_N} := min\left(1.0, 0.7 + 0.3 \cdot \frac{c_{a1}}{1.5 \cdot h_{ef}}\right)$ $\Psi_{ed\_N} = 0.913$ *ACI Eqn. 17.4.2.5b*

Factor for anchor bolts in un-cracked concrete: $\Psi_{c\_N} := 1.4$ *Wall caps are not under load, and per the wall cap design, service moment from post does not cause cracking.* *ACI 17.4.2.6*

Factor for anchor bolts in un-cracked concrete near an edge without supplementary reinforcement: $\Psi_{cp\_N} := min\left(1.0, \max\left(\frac{c_{a1}}{c_{ac}}, \frac{1.5 \cdot h_{ef}}{c_{ac}}\right)\right)$ $\Psi_{cp\_N} = 0.75$ *ACI Eqn. 17.4.2.7b*

Nominal concrete tension breakout strength: $\phi N_{cbg} := \phi_{adh} \cdot \frac{A_{Nc}}{A_{Nco}} \cdot \Psi_{ec\_N} \cdot \Psi_{ed\_N} \cdot \Psi_{c\_N} \cdot \Psi_{cp\_N} \cdot N_b$ $\phi N_{cbg} = 9.382$ ***kip*** *ACI Eqn. 17.4.2.1b*

Check of concrete tension breakout failure:

```
Concrete_Tension_Breakout_Check := if ϕN_cbg ≥ n_ab • T_u_ab
                                       "Bolt is satisfactory."
                                   else
                                       "Bolt is no good."
```

Concrete_Tension_Breakout_Check = "Bolt is satisfactory."

Pullout strength cast-in, post-installed expansion, or undercut anchor in tension (ACI 17.4.3)

Proposed anchors are post-installed adhesive, not headed studs or bolts, expansion anchors, or undercut anchors; so, no check is required.

Concrete side-face blowout strength of headed anchor in tension (ACI 17.4.4)

Proposed anchors are post-installed adhesive, not headed studs or bolts; so, no check is required.

Bond strength of adhesive anchor in tension (ACI 17.4.5)

Minimum bond stress for HY 200 Epoxy per HILTI ESR-3187:

$$\tau_{uncr\_HY\_200} := 0.65 \cdot \left(\frac{f'_c}{2500\ psi}\right)^{0.1} \cdot 2220\ psi = 1512.441\ psi$$

Per HILTI ESR-3187 Table 14, basic un-cracked bond strength is 2,220 psi; this value is factored by a straight 0.65 for either wet or dry installation conditions and by a small boost from concrete strength higher than 2,500 psi

Minimum bond stress for HIT-RE 500 Epoxy per HILTI ESR-3814:

$$\tau_{uncr\_HIT\_RE\_500} := 0.65 \cdot \left(\frac{f'_c}{2500\ psi}\right)^{0.15} \cdot 2210\ psi = 1541.429\ psi$$

Per HILTI ESR-3814 Table 12, basic un-cracked bond strength is 2,210 psi. This value is based on diamond coring and roughening afterwards; it is lower than being hammer-drilled with carbide bit. The socket must be roughened if coring with a diamond bit; this should be written on the plans. Factors are a straight 0.65 reduction factor independent of wet or dry concrete conditions during installation and a small boost for using concrete higher than 2,500 psi. The smaller factor for cracked concrete is used since no supplementary rebar is being provided; this also matches with reduction factor below.

Minimum bond stress strength: $\tau_{uncr} := min\left(\tau_{uncr\_HY\_200}, \tau_{uncr\_HIT\_RE\_500}\right)$ $\tau_{uncr} = 1512.441$ ***psi***

Distance to edge of project influence area: $c_{Na} := 10 \cdot d_a \cdot \sqrt{\frac{\tau_{uncr}}{1100\ psi}}$ $c_{Na} = 7.329$ ***in*** *ACI Eqn. 17.4.5.1d*

Check if anchor bolts act in group for bond failure: *ACI 17.2.1.1*

```
Group_Bond_Failure_Check := if s_1 ≤ 2 c_Na
                                "Bolts act in group."
                            else
                                "Bolts act singly."
```

Group_Bond_Failure_Check = "Bolts act in group."

Theoretical projected influence area of a single bolt far from an edge: $A_{Nao} := (2\ c_{Na})^2$ $A_{Nao} = 214.835\ in^2$ *ACI Eqn. 17.4.5.1c*

Actual projected influence area for bolt(s): $A_{Na} := min\left(\left(c_{Na} + min\left(s_1, 2 \cdot c_{Na}\right) + min\left(c_{Na}, c_{a2}\right)\right) \cdot \left(c_{a1} + c_{Na}\right), n_{ab} \cdot A_{Nao}\right)$ $A_{Na} = 273.826\ in^2$ *ACI Fig. R17.4.5.1*

Basic bond strength of adhesive anchor: $N_{ba} := \tau_{uncr} \cdot \pi \cdot d_a \cdot h_{ef}$ $N_{ba} = 14.848\ kip$ *ACI Eqn. 17.4.5.2*

Concrete is not light weight; so, lambda-a is set to 1.0; per ACI 17.4.5.2, un-cracked bond stress may be used.

Factor for eccentrically loaded anchor bolts: $\Psi_{ec\_Na} := 1.0$ *Anchor bolts are not loaded eccentrically.* *ACI 17.4.5.3*

Factor for anchor bolts near an edge: $\Psi_{ed\_Na} := min\left(1.0, 0.7 + 0.3 \cdot \frac{c_{a1}}{c_{Na}}\right)$ $\Psi_{ed\_Na} = 0.918$ *ACI Eqn. 17.4.5.4b*

Factor for anchor bolts in un-cracked concrete near an edge without supplementary reinforcement: $\Psi_{cp\_Na} := min\left(1.0, \max\left(\frac{c_{a1}}{c_{ac}}, \frac{c_{Na}}{c_{ac}}\right)\right)$ $\Psi_{cp\_Na} = 0.733$ *ACI Eqn. 17.4.5.5b*

Wall caps are not under load, and per the wall cap design, service moment from post does not cause cracking.

Nominal bond strength of the adhesive anchor(s): $\phi N_{ag} := \phi_{adh} \cdot \frac{A_{Na}}{A_{Nao}} \cdot \Psi_{ec\_Na} \cdot \Psi_{ed\_Na} \cdot \Psi_{cp\_Na} \cdot N_{ba}$ $\phi N_{ag} = 8.272\ kip$ *ACI Eqn. 17.4.5.1b*

Check of bolt bond stress failure:

```
Bond_Stress_Check := if ϕN_ag ≥ n_ab • T_u_ab
                        "Bolt is satisfactory."
                     else
                        "Bolt is no good."
```

Bond_Stress_Check = "Bolt is satisfactory."

Steel strength of anchor in shear (17.5.1)

Steel shear strength of anchor is confirmed above; so, no check here is necessary.

Concrete breakout strength of anchor in shear (17.5.2)

Check bolt group action for shear concrete breakout: *ACI 17.2.1.1*

```
Group_Shear_Breakout_Check := if s_1 ≤ 3 • c_a1
                                 "Bolts act in group."
                              else
                                 "Bolts act singly."
```

Group_Shear_Breakout_Check = "Bolts act in group."

Theoretical projected influence area of a single bolt far from an edge: $A_{Vco} := 4.5 \cdot c_{a1}^{\ 2}$ $A_{Vco} = 127.122\ in^2$ *ACI Eqn. 17.5.2.1c*

Actual projected influence area for bolt(s): $A_{Vc} := min\left(1.5 \cdot c_{a1} \cdot \left(1.5 \cdot c_{a1} + min\left(s_1, 3 \cdot c_{a1}\right) + min\left(1.5 \cdot c_{a1}, c_{a2}\right)\right), n_{ab} \cdot A_{Vco}\right)$ $A_{Vc} = 182.929\ in^2$ *ACI Fig. R17.5.2.1b*

Load bearing length: $l_e := h_{ef}$ $l_e = 5\ in$ *ACI 17.5.2.2*

Basic concrete breakout strength in shear for single anchor:

$$V_b := min\left(\left(7\left(\frac{l_e}{d_a}\right)^{0.2} \cdot \sqrt{\frac{d_a}{in}}\right) \cdot 1.0 \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{c_{a1}}{in}\right)^{1.5}, 9 \cdot 1.0 \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{c_{a1}}{in}\right)^{1.5}\right) \cdot lbf \quad V_b = 6.5\ kip$$

Concrete is not light weight; so, lambda-a is set to 1.0. *ACI Eqns. 17.5.2.2a & 17.5.2.2b*

Factor for eccentrically loaded anchor bolts: $\Psi_{ec\_V} := 1.0$ *Anchor bolts are not loaded eccentrically.* *ACI 17.5.2.5*

Factor for anchor bolts near an edge: $\Psi_{ed\_V} := min\left(1.0, 0.7 + 0.3 \cdot \frac{c_{a2}}{1.5 \cdot c_{a1}}\right)$ $\Psi_{ed\_V} = 1$ *ACI Eqns. 17.5.2.6a & 17.5.2.6b*

Factor for anchor bolts in un-cracked concrete: $\Psi_{c\_V} := 1.4$ *Wall caps are not under load, and per the wall cap design, service moment from post does not cause cracking.* *ACI 17.5.2.7*

Factor for small embedment $\Psi_{h\_V} := min\left(1.0, \sqrt{\frac{1.5 \cdot c_{a1}}{h_{ef}}}\right)$ $\Psi_{h\_V} = 1$ *ACI Eqn. 17.5.2.8*

Nominal concrete shear breakout strength: $\phi V_{cbg} := \phi_{adh} \cdot \frac{A_{Vc}}{A_{Vco}} \cdot \Psi_{ec\_V} \cdot \Psi_{ed\_V} \cdot \Psi_{c\_V} \cdot \Psi_{h\_V} \cdot V_b$ $\phi V_{cbg} = 8.512\ \mathbf{kip}$ *ACI Eqn. 17.5.2.1b*

Check of concrete shear breakout failure: $Concrete\_Shear\_Breakout\_Check :=$ if $\phi V_{cbg} \geq V_{post\_u}$
"Bolt is satisfactory."
else
"Bolt is no good."

$Concrete\_Shear\_Breakout\_Check =$ "Bolt is satisfactory."

Concrete pryout strength of anchor in shear (17.5.3)

Basic concrete pryout strength of a single anchor in shear: $\phi N_{cpg} := min\left(\phi N_{ag}, \phi N_{cbg}\right)$ $\phi N_{cpg} = 8.272\ \mathbf{kip}$ *ACI 17.5.3.1*

Concrete pryout strength in shear coefficient: $k_{cp} :=$ if $h_{ef} < 2.5\ \mathbf{in}$
1.0
else
2.0

$k_{cp} = 2$ *ACI 17.5.3.1*

Nominal concrete pryout strength of anchor(s) in shear: $\phi V_{cpg} := k_{cp} \cdot \phi N_{cpg}$ $\phi V_{cpg} = 16.545\ \mathbf{kip}$ *ACI Eqn. 17.5.3.1b*

Check of concrete pryout strength in shear: $Concrete\_Shear\_Pryout\_Check :=$ if $\phi V_{cpg} \geq V_{post\_u}$
"Bolt is satisfactory."
else
"Bolt is no good."

$Concrete\_Shear\_Pryout\_Check =$ "Bolt is satisfactory."

Vertical Interior Post and Horizontal Rail Design

Given:

Post Height:	$H_{post} := 66\ in$	*Plans*
Step Height:	$H_{step} := 24\ in$	
Post and Rail Yield Strength:	$F_y := 50\ ksi$	*ASTM F1043*
Post and Rail Modulus of Elasticity:	$E_s := 29000\ ksi$	
Post and Rail Ultimate Strength:	$F_u := 58\ ksi$	

Properties for ASTM F1043 IC 2-3/8" Pipe for Interior Posts

Post OD:	$OD_{post} := 2.375\ in$
Post Thickness:	$t_{post} := 0.13\ in$

Properties for ASTM F1043 IC 1-5/8" Pipe for Rails

Rail OD:	$OD_{rail} := 1.66\ in$
Rail Thickness:	$t_{rail} := 0.111\ in$

Design Point Live Load	$P_{LL} := 200\ lbf$	*AASHTO 13.8.2*
Design Uniform Live Load	$w_{LL} := 0\ plf$	*AASHTO 13.8.2*
Post spacing:	$L_{spc} := 8\ ft$	*Plans*
Weight of chain link fence:	$f_{clf} := 0.48\ psf$	
Design wind load from chain link fence:	$f_{wind} := 15\ psf$	*AASHTO 13.8.2*

Load Factors (AASHTO Tbl. 3.4.1-1):

PL Load Factor:	$\gamma_{PL} := 1.75$
DC Load Factor:	$\gamma_{DL} := 1.25$
WS Load Factor:	$\gamma_{WS} := 1.00$

Resistance Factors:

Steel Flexure (AASHTO 6.5.4.2):	$\phi_f := 1.00$
Steel Shear (AASHTO 6.5.4.2):	$\phi_v := 1.00$
Tension, Yielding in Gross Section:	$\phi_y := 0.95$
Bending (AISC F1):	$\phi_b := 0.90$
Shear (AISC G1):	$\phi_{v\_AISC} := 0.90$
Bearing (AISC DG#1):	$\phi_{brg} := 0.60$
Fillet Weld (AISC Tbl. J2.5):	$\phi_{fw} := 0.75$
Bolts (AISC J3.6 & J3.7):	$\phi_{ab} := 0.75$
Adhesive Anchor Bolts (ACI 17.3.3, Condition B, Category 1):	$\phi_{adh} := 0.65$

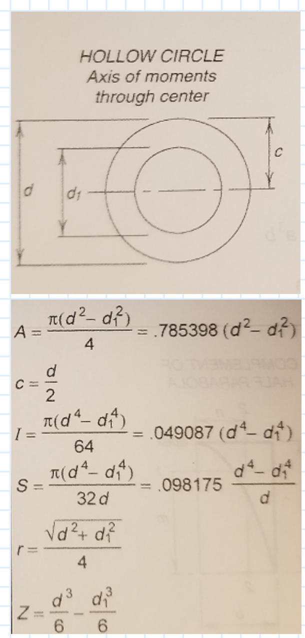
Steel weight density: $\gamma_{steel} := 490\ pcf$

ASTM F1043 Group IC Electric Resistant Welded 50,000 psi yield steel pipe																	
Trade Reference	Decimal O.D. Equivalent		Pipe wall Thickness		Weight		Section Modulus			Min. Yield Strength			Max Bending Moment	Calculated Load (lbs)			
									x			=		10' Free	Cantilever		
O.D.	inches	(mm)	inches	(mm)	lb./ft.	(kg/m)	inches³	(mm³)	x	psi	(Mpa)	=	Lb.In.	Supported	4'	6'	
1 5/8"	1.660	42.16	0.111	2.82	1.84	2.74	0.1962	4.98	x	50000	345	=	9810	327	204	136	
1 7/8"	1.900	48.26	0.120	3.05	2.28	3.39	0.2810	7.14	x	50000	345	=	14050	468	293	195	
2 3/8"	2.375	60.33	0.130	3.30	3.12	4.64	0.4881	12.40	x	50000	345	=	24405	814	508	339	
2 7/8"	2.875	73.03	0.160	4.06	4.64	6.91	0.8778	22.30	x	50000	345	=	43890	1463	914	610	
3 1/2"	3.500	88.90	0.160	4.06	5.71	8.50	1.3408	34.06	x	50000	345	=	67042	2235	1397	931	
4"	4.000	101.60	0.160	4.06	6.57	9.78	1.7820	45.26	x	50000	345	=	89098	2970	1856	1237	
4 1/2"	4.500	114.30	0.160	4.14	7.42	11.04	2.2859	57.99	X	50000	345	=	114295	3810	5486	1587	

Output:

Post Section Properties:

Post inside diameter:	$ID_{post} := OD_{post} - 2 \cdot t_{post}$	$ID_{post} = 2.115\ in$
Post Area:	$A_{post} := 0.785398 \cdot \left(OD_{post}^2 - ID_{post}^2\right)$	$A_{post} = 0.917\ in^2$
Post Unit Weight:	$w_{post} := \gamma_{steel} \cdot A_{post}$	$w_{post} = 3.12\ plf$
Post centroid:	$c_{post} := 0.5 \cdot OD_{post}$	$c_{post} = 1.188\ in$
Post Moment of Inertial:	$I_{post} := 0.049087 \cdot \left(OD_{post}^4 - ID_{post}^4\right)$	$I_{post} = 0.58\ in^4$
Post Section Modulus:	$S_{post} := \frac{I_{post}}{c_{post}}$	$S_{post} = 0.488\ in^3$
Post Plastic Section Modulus:	$Z_{post} := \frac{OD_{post}^3 - ID_{post}^3}{6}$	$Z_{post} = 0.656\ in^3$



Rail Section Properties:

Rail inside diameter:	$ID_{rail} := OD_{rail} - 2 \cdot t_{rail}$	$ID_{rail} = 1.438\ in$
Rail Area:	$A_{rail} := 0.785398 \cdot \left(OD_{rail}^2 - ID_{rail}^2\right)$	$A_{rail} = 0.54\ in^2$
Rail Unit Weight:	$w_{rail} := \gamma_{steel} \cdot A_{rail}$	$w_{rail} = 1.838\ plf$
Rail centroid:	$c_{rail} := 0.5 \cdot OD_{rail}$	$c_{rail} = 0.83\ in$
Rail Moment of Inertial:	$I_{rail} := 0.049087 \cdot \left(OD_{rail}^4 - ID_{rail}^4\right)$	$I_{rail} = 0.163\ in^4$
Rail Section Modulus:	$S_{rail} := \frac{I_{rail}}{c_{rail}}$	$S_{rail} = 0.196\ in^3$
Rail Plastic Section Modulus:	$Z_{rail} := \frac{OD_{rail}^3 - ID_{rail}^3}{6}$	$Z_{rail} = 0.267\ in^3$

Post concentrated live load applied at high top rail: $P_{post\_LL\_H} := P_{LL} + w_{LL} \cdot \frac{L_{spc}}{2} = 0.2\ kip$ $P_{post\_LL\_H} = 0.2\ kip$ *AASHTO Eqn. 13.8.2-1, modified for split top rails*

Post concentrated live load applied at low top rail: $P_{post\_LL\_L} := w_{LL} \cdot \frac{L_{spc}}{2} = 0\ kip$ $P_{post\_LL\_L} = 0\ kip$

Post moment loading from live load: $M_{post\_LL} := P_{post\_LL\_H} \cdot H_{post} + P_{post\_LL\_L} \cdot (H_{post} - H_{step})$ $M_{post\_LL} = 13200\ lbf \cdot in$ *Post treated as cantilevered beam*

Post shear from live load: $V_{post\_LL} := P_{post\_LL\_H} + P_{post\_LL\_L}$ $V_{post\_LL} = 0.2\ kip$

Rail moment from live load applied: $M_{rail\_LL} := \sqrt{2} \cdot \frac{w_{LL} \cdot L_{spc}^2}{8} + \frac{P_{LL} \cdot L_{spc}}{4}$ $M_{rail\_LL} = 4800\ lbf \cdot in$ *Rail treated as simply supported beam with vertical and horizontal live loads combined into resultant direction.*

Rail moment from dead load: $M_{rail\_DL} := \frac{w_{rail} \cdot L_{spc}^2}{8} + \frac{f_{clf} \cdot \frac{H_{post}}{2} \cdot L_{spc}^2}{8}$ $M_{rail\_DL} = 303.173\ lbf \cdot in$

Rail shear from live load: $V_{rail\_LL} := \sqrt{2} \cdot \frac{w_{LL} \cdot L_{spc}}{2} + \frac{P_{LL}}{2}$ $V_{rail\_LL} = 0.1\ kip$

Rail shear from dead load: $V_{rail\_DL} := \frac{w_{rail} \cdot L_{spc}}{2} + \frac{f_{clf} \cdot \frac{H_{post}}{2} \cdot L_{spc}}{2}$ $V_{rail\_DL} = 0.013\ kip$

Factored Shear Load on Post: $V_{post\_u} := \gamma_{PL} \cdot V_{post\_LL}$ $V_{post\_u} = 0.35\ kip$

Factored Moment Load on Post: $M_{post\_u} := \gamma_{PL} \cdot M_{post\_LL}$ $M_{post\_u} = 23100\ lbf \cdot in$

AASHTO load factors used instead of ASCE load factors found in AISC and ACI. This is acceptable as it is more conservative.

Factored Shear Load on Rail: $V_{rail\_u} := \gamma_{PL} \cdot V_{rail\_LL} + \gamma_{DL} \cdot V_{rail\_DL}$ $V_{rail\_u} = 0.191\ kip$

Factored Moment Load on Rail: $M_{rail\_u} := \gamma_{PL} \cdot M_{rail\_LL} + \gamma_{DL} \cdot M_{rail\_DL}$ $M_{rail\_u} = 8778.966\ lbf \cdot in$

Vertical dead load was combined directly with live load resultant since it was so small compared to the live load.

Post Analysis:

Following AASHTO 6.12.1.2.3c for Shear Design:

Gross Area: $A_g := A_{post}$ $A_g = 0.917\ in^2$

Distance from Max to 0 Shear: $L_v := H_{post}$ $L_v = 66\ in$

Critical Strength for Shear: $F_{cr} := min\left(0.58 \cdot F_y, \max\left(\frac{1.6 \cdot E_s}{\sqrt{\frac{L_v}{OD_{post}}} \left(\frac{OD_{post}}{t_{post}}\right)^{\frac{5}{4}}}, \frac{0.78 \cdot E_s}{\left(\frac{OD_{post}}{t_{post}}\right)^{\frac{3}{2}}}\right)\right)$ $F_{cr} = 29\ ksi$ *AASHTO Eqns. 6.12.1.2.3c-2 & 6.12.1.2.3c-3*

Factored nominal shear resistance: $\phi V_n := \phi_v \cdot 0.5\ F_{cr} \cdot A_g$ $\phi V_n = 13.295\ kip$ *AASHTO Eqn. 6.12.1.2.3c-1*

Post Shear Check: $\frac{\phi V_n}{V_{post\_u}} = 37.985$

$Post\_Shear\_Check :=$ if $\frac{\phi V_n}{V_{post\_u}} \geq 1.0$
 "Post shear strength is satisfactory."
else
 "Post is not satisfactory."

$Post\_Shear\_Check =$ "Post shear strength is satisfactory."

Following AASHTO 6.12.2.2.3 for Flexure Design:

Check of Noncompact Section: $Check\_Compact :=$ if $\frac{OD_{post}}{t_{post}} \leq \frac{0.07 \cdot E_s}{F_y}$
 "Section is compact. Local buckling does not apply."
else
 "Section is not compact. Check wall slenderness."

Per AASHTO 6.12.2.2.3, as long D/t does not exceed 0.07E/Fy, plastic modulus and equation 6.12.2.2.3-1 may be used.

$Check\_Compact =$ "Section is compact. Local buckling does not apply."

Factored Nominal Moment Resistance: $\phi M_n := \phi_f \cdot F_y \cdot Z_{post}$ $\phi M_n = 32.797\ kip \cdot in$ AASHTO Eqn. 6.12.2.2.3-1

Post Flexural Check: $\dfrac{\phi M_n}{M_{post\_u}} = 1.42$

$Post\_Flex\_Check :=$ if $\dfrac{\phi M_n}{M_{post\_u}} \geq 1.0$
  "Post flexural strength is satisfactory."
else
  "Post is not satisfactory."

$Post\_Flex\_Check =$ "Post flexural strength is satisfactory."

Rail Analysis:

Following AASHTO 6.12.1.2.3c for Shear Design:

Gross Area: $A_g := A_{rail}$ $A_g = 0.54\ in^2$

Distance from Max to 0 Shear: $L_v := \dfrac{L_{spc}}{2}$ $L_v = 48\ in$

Critical Strength for Shear: $F_{cr} := min\left(0.58 \cdot F_y, \max\left(\dfrac{1.6 \cdot E_s}{\sqrt{\dfrac{L_v}{OD_{rail}}}\left(\dfrac{OD_{rail}}{t_{rail}}\right)^{\frac{5}{4}}}, \dfrac{0.78 \cdot E_s}{\left(\dfrac{OD_{rail}}{t_{rail}}\right)^{\frac{3}{2}}}\right)\right)$ $F_{cr} = 29\ ksi$ AASHTO Eqns. 6.12.1.2.3c-2 & 6.12.1.2.3c-3

Factored Nominal Shear Resistance: $\phi V_n := \phi_v \cdot 0.5\ F_{cr} \cdot A_g$ $\phi V_n = 7.832\ kip$ AASHTO Eqn. 6.12.1.2.3c-1

Rail Shear Check: $\dfrac{\phi V_n}{V_{rail\_u}} = 41.052$

$Rail\_Shear\_Check :=$ if $\dfrac{\phi V_n}{V_{rail\_u}} \geq 1.0$
  "Rail shear strength is satisfactory."
else
  "Rail is not satisfactory."

$Rail\_Shear\_Check =$ "Rail shear strength is satisfactory."

Following AASHTO 6.12.2.2.3 for Flexure Design:

Check of Noncompact Section:

$Check\_Compact :=$ if $\dfrac{OD_{rail}}{t_{rail}} \leq \dfrac{0.07 \cdot E_s}{F_y}$
  "Section is compact. Local buckling does not apply."
else
  "Section is not compact. Check wall slenderness."

Per AASHTO 6.12.2.2.3, as long D/t does not exceed 0.07E/Fy, plastic modulus and equation 6.12.2.2.3-1 may be used.

$Check\_Compact =$ "Section is compact. Local buckling does not apply."

Factored Nominal Moment Resistance: $\phi M_n := \phi_f \cdot F_y \cdot Z_{rail}$ $\phi M_n = 13.339\ kip \cdot in$ AASHTO Eqn. 6.12.2.2.3-1

Post Flexural Check: $\dfrac{\phi M_n}{M_{rail\_u}} = 1.519$

$Rail\_Flex\_Check :=$ if $\dfrac{\phi M_n}{M_{rail\_u}} \geq 1.0$
  "Rail flexural strength is satisfactory."
else
  "Rail is not satisfactory."

$Rail\_Flex\_Check =$ "Rail flexural strength is satisfactory."

Confirming that Wind Loading Doesn't Control:

Per last paragraph of AASHTO 13.8.2, the wind load on the chain link fence is not applied simultaneously with the live load.

Uniform wind load on post: $w_{post\_wind} := f_{wind} \cdot L_{spc}$ $w_{post\_wind} = 120\ plf$

Design moment from wind on post: $M_{post\_wind\_u} := \gamma_{WS} \cdot \frac{w_{post\_wind} \cdot H_{post}^2}{2}$ $M_{post\_wind\_u} = 21780\ lbf \cdot in$

$M_{post\_u} = 23100\ lbf \cdot in$ <- LL controls

Design shear from wind on post: $V_{post\_wind\_u} := \gamma_{WS} \cdot w_{post\_wind} \cdot H_{post}$ $V_{post\_wind\_u} = 0.66\ kip$

$V_{post\_u} = 0.35\ kip$ <- LL controls

Uniform wind on rail: $w_{rail\_wind} := f_{wind} \cdot \frac{H_{post}}{2}$ $w_{rail\_wind} = 41.25\ plf$

Design moment from wind on rail: $M_{rail\_wind\_u} := \gamma_{WS} \cdot \frac{w_{rail\_wind} \cdot L_{spc}^2}{8}$ $M_{rail\_wind\_u} = 3960\ lbf \cdot in$

$M_{rail\_u} = 8778.966\ lbf \cdot in$ <- LL controls

Design shear from wind on rail: $V_{rail\_wind\_u} := \gamma_{WS} \cdot w_{rail\_wind} \cdot \frac{L_{spc}}{2}$ $V_{rail\_wind\_u} = 0.165\ kip$

$V_{rail\_u} = 0.191\ kip$ <- LL controls

Base Plate Design - Line Post w/ Axial Compression

Given: *Plans*

Cap width: $W_{cap} := 15.63\ in$

Distance from post to end of cap: $L_{end} := 8\ in$

Plate thickness: $t_p := .5\ in$

Plate length (perpendicular to fence): $N_{plate} := 8\ in$

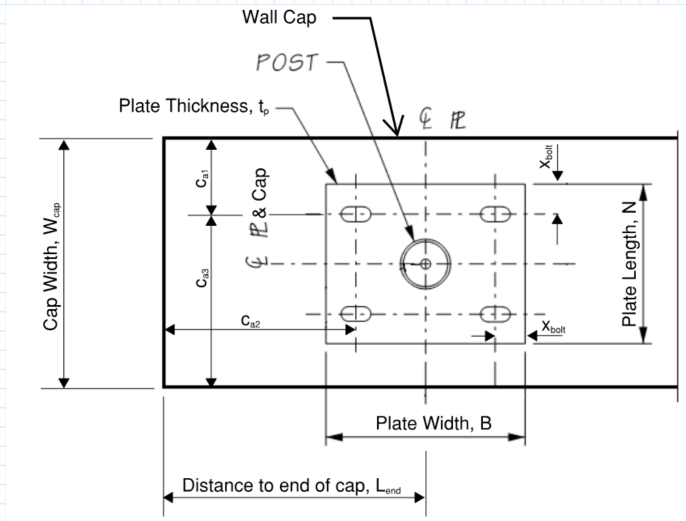
Plate width (parallel to fence): $B_{plate} := 10\ in$

Compressive Strength of Concrete: $f'_c := 4\ ksi$

Side clearance to anchor bolts: $x_{bolt} := 1.5\ in$

Base plate steel yield strength: $F_{y\_plate} := 36\ ksi$

Number of rails: $n_{rail} := 4$



Output:

Plate Area: $A_{plate} := N_{plate} \cdot B_{plate}$ $\quad A_{plate} = 80\ in^2$

Distance from bolt to near face of cap: $c_{a1} := \frac{1}{2}\left(W_{cap} - N_{plate}\right) + x_{bolt}$ $\quad c_{a1} = 5.315\ in$

Distance from outside bolt to end of cap: $c_{a2} := L_{end} - \frac{B_{plate}}{2} + x_{bolt}$ $\quad c_{a2} = 4.5\ in$

Distance from bolt to far face of cap: $c_{a3} := W_{cap} - c_{a1}$ $\quad c_{a3} = 10.315\ in$

Bearing Area taken to Be Same as Plate Area: $A_{bearing} := A_{plate}$ $\quad A_{bearing} = 80\ in^2$ *Conservatively setting bearing area to the same as the plate.*

Max allowed bearing pressure: $f_{pu\_max} := \phi_{brg} \cdot min\left(0.85 \cdot f'_c \cdot \sqrt{\frac{A_{bearing}}{A_{plate}}}, 1.7 \cdot f'_c\right)$ $\quad f_{pu\_max} = 2.04\ ksi$ *ACI Tbl. 14.5.6.1*

Max allowed bearing pressure line: $q_{max} := f_{pu\_max} \cdot B_{plate}$ $\quad q_{max} = \left(2.448 \cdot 10^5\right) \frac{lbf}{ft}$

Post dead load on plate $P_{post\_DL} := w_{post} \cdot H_{post}$ $\quad P_{post\_DL} = 0.017\ kip$

Rail dead load on plat: $P_{rail\_DL} := n_{rail} \cdot 2 \cdot V_{rail\_DL}$ $\quad P_{rail\_DL} = 0.101\ kip$

Factored vertical load on plate: $P_u := \gamma_{DL} \cdot \left(P_{post\_DL} + P_{rail\_DL}\right)$ $\quad P_u = 0.148\ kip$

Minimum length of area of bearing: $Y_{min} := \frac{P_u}{q_{max}}$ $\quad Y_{min} = 0.007\ in$ *AISC DG#1 Eqn. 3.3.3*

Critical eccentricity distance: $e_{crit} := \frac{N_{plate}}{2} - \frac{Y_{min}}{2}$ $\quad e_{crit} = 3.996\ in$ *AISC DG#1 Eqn. 3.3.7*

Eccentricity of loading: $e_{loading} := \frac{M_{post\_u}}{P_u}$ $\quad e_{loading} = 156.322\ in$ *AISC DG#1 Eqn. 3.3.6*

Small moment check: $Small\_Moment\_Check :=$ if $e_{loading} \le e_{crit}$
  "Moment is small, no need for anchor bolts."
else
  "Moment is large, need anchor bolts."

$Small\_Moment\_Check =$ "Moment is large, need anchor bolts."

Distance from bolt to center of post: $f_{dim} := \frac{N_{plate}}{2} - x_{bolt}$ $\quad f_{dim} = 2.5\ in$

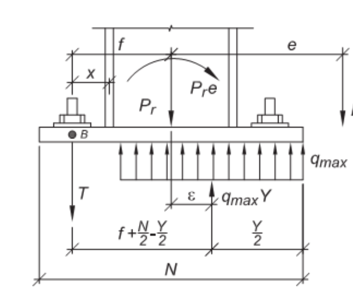


Fig. 3.4.1. Base plate with large moment.

Plate dimension check

$$Plate\_Dim\_Check := \text{if } \left(f_{dim} + \frac{N_{plate}}{2}\right)^2 \geq \frac{2 \cdot P_u \cdot \left(e_{loading} + f_{dim}\right)}{q_{max}} \;\; \|\text{“Plate dimensions are OK.”} \;\; \text{else} \;\; \|\text{“Plate needs to be longer and/or wider.”}$$

$Plate\_Dim\_Check =$ “Plate dimensions are OK.”

Length of bearing area centered at the eccentricity of this loading:

$$Y_{loading} := \left(f_{dim} + \frac{N_{plate}}{2}\right) - \sqrt{\left(f_{dim} + \frac{N_{plate}}{2}\right)^2 - \frac{2 \cdot P_u \cdot \left(e_{loading} + f_{dim}\right)}{q_{max}}}$$

$Y_{loading} = 0.179\ in$

AISC DG#1 Eqn. 3.4.3

Required tensile resistance in anchor rods:

$$T_u := q_{max} \cdot Y_{loading} - P_u$$

$T_u = 3.513\ kip$ *AISC DG#1 Eqn. 3.4.2*

Find minimum required thickness for plate based on bending at bearing interface:

Find bearing bending line distance from edge of plate (AISC DG#1, 3.1.3):

$$m_{plate} := \frac{N_{plate} - 0.8\ OD_{post}}{2}$$

$m_{plate} = 3.05\ in$

Calculating minimum thickness based on bearing:

$$t_{p\_brng\_req} := \text{if } Y_{loading} \geq m_{plate} \;\; \left\| 1.5 \cdot m_{plate} \cdot \sqrt{\frac{f_{pu\_max}}{F_{y\_plate}}} \right. \;\; \text{else} \;\; \left\| 2.11 \cdot \sqrt{\frac{f_{pu\_max} \cdot Y_{loading} \cdot \left(m_{plate} - \frac{Y_{loading}}{2}\right)}{F_{y\_plate}}} \right.$$

$t_{p\_brng\_req} = 0.366\ in$

AISC DG#1 Eqns. 3.3.14a-2 & 3.3.15a-2

Find minimum required thickness for plate based on bending at tension interface:

Find tension bending line distance from edge of plate (AISC DG#1, 3.1.3):

$$x_{ten} := f_{dim} - \frac{0.8 \cdot OD_{post}}{2}$$

$x_{ten} = 1.55\ in$

Calculating minimum thickness based on tension:

$$t_{p\_ten\_req} := 2.11 \cdot \sqrt{\frac{T_u \cdot x_{ten}}{B_{plate} \cdot F_{y\_plate}}}$$

$t_{p\_ten\_req} = 0.26\ in$ *AISC DG#1 Eqn. 3.4.7a*

Controlling minimum required base plate thickness:

$$t_{p\_req} := \max\left(t_{p\_brng\_req}, t_{p\_ten\_req}\right)$$

$t_{p\_req} = 0.366\ in$

Check chosen plate thickness:

$$Plate\_Thick\_Check := \text{if } t_p \geq t_{p\_req} \;\; \|\text{“Chosen plate thickness is adequate.”} \;\; \text{else} \;\; \|\text{“Need a thicker plate.”}$$

$Plate\_Thick\_Check =$ “Chosen plate thickness is adequate.”

Pipe to Plate Fillet Weld Connection Design

Given:

Minimum Fillet Weld Size: $w_{min} := \frac{1}{8}\ in$ — *Min fillet weld size based on AISC Table J2-4*

Chosen fillet weld size: $w := \frac{5}{16}\ in$

Weld material: $F_{EXX} := 70\ ksi$

Output:

Welded Connection to Base Plate Design:

Gross Length of Weld is Post Perimeter: $L_g := \pi \cdot OD_{post}$ $\quad L_g = 7.461\ in$

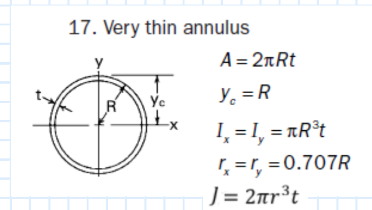
Effective Length of Weld: $L_w := L_g - 2 \cdot w$ $\quad L_w = 6.836\ in$

Effective Throat Thickness: $t_e := min\left(w \cdot \sin(45\ deg), \frac{L_w}{4}\right)$ $\quad t_e = 0.221\ in$ — *AISC, Sect. J2, Pts. 2a & 2b*

Area of Weld: $A_w := L_w \cdot t_e$ $\quad A_w = 1.511\ in^2$ — *AISC, Sect. J2, Pts. 2a*

Moment of Inertia of circular fillet weld: $I_w := \pi \cdot \left(\frac{OD_{post}}{2}\right)^3 \cdot t_e$ $\quad I_w = 1.162\ in^4$

Polar moment of Inertia of circular fillet weld: $J_w := 2\ \pi \cdot \left(\frac{OD_{post}}{2}\right)^3 \cdot t_e$ $\quad J_w = 2.325\ in^4$



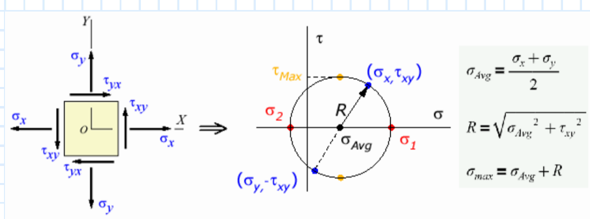
Determine design strength of weld:

Nominal strength of weld metal: $F_w := \phi_{fw} \cdot 0.6 \cdot F_{EXX}$ $\quad F_w = 31.5\ ksi$ — *AISC, Tbl. J2.5*

Normal stress caused by bending moment: $\sigma_b := \frac{M_{post\_u} \cdot \left(\frac{OD_{post}}{2}\right)}{I_w}$ $\quad \sigma_b = 23.597\ ksi$ $\quad \sigma = \frac{M}{S} = \frac{M \cdot c}{I}$

Stress caused by shearing force: $\tau_v := \frac{V_{post\_u}}{A_w}$ $\quad \tau_v = 0.232\ ksi$

Resultant stress in weld from loading: $\sigma_{max} := \frac{\sigma_b}{2} + \sqrt{\left(\frac{\sigma_b}{2}\right)^2 + \tau_v^{\ 2}}$ $\quad \sigma_{max} = 23.599\ ksi$

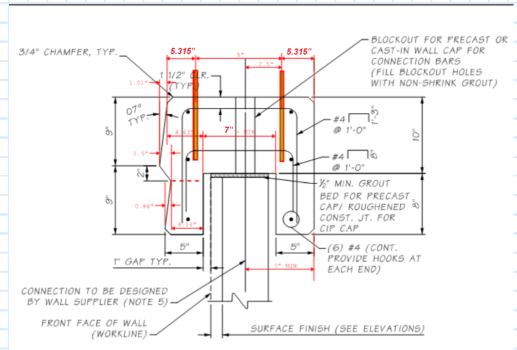


Check of weld thickness:

$Weld\_Design\_Check :=$ if $F_w \geq \sigma_{max}$
  "Chosen weld size is sufficient."
else
  "Need bigger fillet weld."

$Weld\_Design\_Check =$ "Chosen weld size is sufficient."

Anchor Bolt Connection Design



WALL CAP TYPICAL DETAIL

Given:

Number of anchor bolts resisting loads:	$n_{ab} := 2$	*Only one side's bolts resist tension or shear.*
Bolts are specified as ASTM F1554 and Grade A36		
Bolt diameter:	$d_{ab} := \frac{5}{8}\ in$	*Plans*
Bolt area:	$A_b := 0.307\ in^2$	*AISC Tbl. 7-18*
Bolt nominal yield stress strength:	$F_{y\_bolt} := 36\ ksi$	*AISC Tbl. 2-3*
Bolt nominal ultimate tensile stress strength:	$F_{u\_bolt} := 58\ ksi$	
Bolt embedment:	$h_{ef} := 5\ in$	*Plans*

Output:

Tension anchor bolt spacing: $s_1 := \dfrac{B_{plate} - 2 \cdot x_{bolt}}{n_{ab} - 1}$ $\quad s_1 = 7\ in$

Bolt nominal tensile stress strength: $F_{nt} := 0.75 \cdot F_{u\_bolt}$ $\quad F_{nt} = 43.5\ ksi$ *AISC Tbl. J3.2*

Bolt nominal shear stress strength: $F_{nv} := 0.40 \cdot F_{u\_bolt}$ $\quad F_{nv} = 23.2\ ksi$ *AISC Tbl. J3.2, assuming threads within shear plane*

Ultimate tension load on one anchor bolt: $T_{u\_ab} := \dfrac{T_u}{n_{ab}}$ $\quad T_{u\_ab} = 1.757\ kip$

Required shear stress on one bolt: $f_v := \dfrac{V_{post\_u}}{n_{ab} \cdot A_b}$ $\quad f_v = 0.57\ ksi$

Bolt modified nominal tensile stress strength, modified for effects of shear stress: $F_{nt}' := min\left(F_{nt}, 1.3 \cdot F_{nt} - \dfrac{F_{nt}}{\phi_{ab} \cdot F_{nv}} \cdot f_v\right)$ $\quad F_{nt}' = 43.5\ ksi$ *AISC Eqn. J3-3a*

Bolt factored tensile resistance: $\phi R_{n\_bolt} := \phi_{ab} \cdot F_{nt}' \cdot A_b$ $\quad \phi R_{n\_bolt} = 10.016\ kip$ *AISC Eqn. J3-2*

Check of bolt tensile stress:

$Bolt\_Tensile\_Check :=$ if $\phi R_{n\_bolt} \geq T_{u\_ab}$
  "Bolt is satisfactory."
else
  "Bolt is no good."

$Bolt\_Tensile\_Check =$ "Bolt is satisfactory."

Continuing Anchor Bolt Connection Design per ACI 318

Outside diameter of anchor: $d_a := d_{ab}$ $\quad d_a = 0.625\ in$

Critical edge distance for adhesive anchors: $c_{ac} := 2\ h_{ef}$ $\quad c_{ac} = 10\ in$ *ACI 17.7.6*

Steel strength of anchor in tension (ACI 17.4.1)

Steel tension strength of anchor is confirmed above; so, no check here is necessary.

Concrete breakout strength of anchor in tension (ACI 17.4.2)

Check bolt group action for tension concrete breakout: *ACI 17.2.1.1*

$Group\_Tension\_Breakout\_Check :=$ if $s_1 \leq 3 \cdot h_{ef}$
  "Bolts act in group."
else
  "Bolts act singly."

$Group\_Tension\_Breakout\_Check =$ "Bolts act in group."

Theoretical projected influence area of a single bolt far from an edge: $A_{Nco} := 9 \cdot h_{ef}^2$ $\quad A_{Nco} = 225\ in^2$ *ACI Eqn. 17.4.2.1c*

Actual projected influence area for bolt(s): $A_{Nc} := min\left(\left(c_{a1} + 1.5 \cdot h_{ef}\right) \cdot \left(1.5 \cdot h_{ef} + min\left(s_1, 3 \cdot h_{ef}\right) + min\left(1.5 \cdot h_{ef}, c_{a2}\right)\right), n_{ab} \cdot A_{Nco}\right)$ $\quad A_{Nc} = 243.485\ in^2$ *ACI Fig. R17.4.2.1*

Concrete tension breakout strength coefficient: $k_c := 17$ *Value of 17 for post-installed anchors, per ACI 17.4.2.2*

Basic concrete tension breakout strength for single anchor: $N_b := k_c \cdot 1.0 \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{h_{ef}}{in}\right)^{1.5} \cdot lbf$ $N_b = 12.021\ kip$ *ACI Eqn. 17.4.2.2a*

Factor for eccentrically loaded anchor bolts: $\Psi_{ec\_N} := 1.0$ *Anchor bolts are not loaded eccentrically.* *ACI 17.4.2.4*

Factor for anchor bolts near an edge: $\Psi_{ed\_N} := min\left(1.0, 0.7 + 0.3 \cdot \frac{c_{a1}}{1.5 \cdot h_{ef}}\right)$ $\Psi_{ed\_N} = 0.913$ *ACI Eqn. 17.4.2.5b*

Factor for anchor bolts in un-cracked concrete: $\Psi_{c\_N} := 1.4$ *Wall caps are not under load, and per the wall cap design, service moment from post does not cause cracking.* *ACI 17.4.2.6*

Factor for anchor bolts in un-cracked concrete near an edge without supplementary reinforcement: $\Psi_{cp\_N} := min\left(1.0, \max\left(\frac{c_{a1}}{c_{ac}}, \frac{1.5 \cdot h_{ef}}{c_{ac}}\right)\right)$ $\Psi_{cp\_N} = 0.75$ *ACI Eqn. 17.4.2.7b*

Nominal concrete tension breakout strength: $\phi N_{cbg} := \phi_{adh} \cdot \frac{A_{Nc}}{A_{Nco}} \cdot \Psi_{ec\_N} \cdot \Psi_{ed\_N} \cdot \Psi_{c\_N} \cdot \Psi_{cp\_N} \cdot N_b$ $\phi N_{cbg} = 8.102\ kip$ *ACI Eqn. 17.4.2.1b*

Check of concrete tension breakout failure:

```
Concrete_Tension_Breakout_Check := if φNcbg ≥ nab • Tu_ab
                                       "Bolt is satisfactory."
                                   else
                                       "Bolt is no good."
```

Concrete_Tension_Breakout_Check = "Bolt is satisfactory."

Pullout strength cast-in, post-installed expansion, or undercut anchor in tension (ACI 17.4.3)

Proposed anchors are post-installed adhesive, not headed studs or bolts, expansion anchors, or undercut anchors; so, no check is required.

Concrete side-face blowout strength of headed anchor in tension (ACI 17.4.4)

Proposed anchors are post-installed adhesive, not headed studs or bolts; so, no check is required.

Bond strength of adhesive anchor in tension (ACI 17.4.5)

Minimum bond stress for HY 200 Epoxy per HILTI ESR-3187:

$$\tau_{uncr\_HY\_200} := 0.65 \cdot \left(\frac{f'_c}{2500\ psi}\right)^{0.1} \cdot 2220\ psi = 1512.441\ psi$$

Per HILTI ESR-3187 Table 14, basic un-cracked bond strength is 2,220 psi; this value is factored by a straight 0.65 for either wet or dry installation conditions and by a small boost from concrete strength higher than 2,500 psi

Minimum bond stress for HIT-RE 500 Epoxy per HILTI ESR-3814:

$$\tau_{uncr\_HIT\_RE\_500} := 0.65 \cdot \left(\frac{f'_c}{2500\ psi}\right)^{0.15} \cdot 2210\ psi = 1541.429\ psi$$

Per HILTI ESR-3814 Table 12, basic un-cracked bond strength is 2,210 psi. This value is based on diamond coring and roughening afterwards; it is lower than being hammer-drilled with carbide bit. The socket must be roughened if coring with a diamond bit; this should be written on the plans. Factors are a straight 0.65 reduction factor independent of wet or dry concrete conditions during installation and a small boost for using concrete higher than 2,500 psi. The smaller factor for cracked concrete is used since no supplementary rebar is being provided; this also matches with reduction factor below.

Minimum bond stress strength: $\tau_{uncr} := min\left(\tau_{uncr\_HY\_200}, \tau_{uncr\_HIT\_RE\_500}\right)$ $\tau_{uncr} = 1512.441\ psi$

Distance to edge of project influence area: $c_{Na} := 10 \cdot d_a \cdot \sqrt{\frac{\tau_{uncr}}{1100\ psi}}$ $c_{Na} = 7.329\ in$ *ACI Eqn. 17.4.5.1d*

Check if anchor bolts act in group for bond failure: *ACI 17.2.1.1*

```
Group_Bond_Failure_Check := if s1 ≤ 2 cNa
                                "Bolts act in group."
                            else
                                "Bolts act singly."
```

Group_Bond_Failure_Check = "Bolts act in group."

Theoretical projected influence area of a single bolt far from an edge: $A_{Nao} := (2\ c_{Na})^2$ $A_{Nao} = 214.835\ in^2$ ACI Eqn. 17.4.5.1c

Actual projected influence area for bolt(s): $A_{Na} := min\left(\left(c_{Na} + min\left(s_1, 2 \cdot c_{Na}\right) + min\left(c_{Na}, c_{a2}\right)\right) \cdot \left(c_{a1} + c_{Na}\right), n_{ab} \cdot A_{Nao}\right)$ $A_{Na} = 238.062\ in^2$ ACI Fig. R17.4.5.1

Basic bond strength of adhesive anchor: $N_{ba} := \tau_{uncr} \cdot \pi \cdot d_a \cdot h_{ef}$ $N_{ba} = 14.848\ kip$ ACI Eqn. 17.4.5.2

Concrete is not light weight; so, lambda-a is set to 1.0; per ACI 17.4.5.2, un-cracked bond stress may be used.

Factor for eccentrically loaded anchor bolts: $\Psi_{ec\_Na} := 1.0$ *Anchor bolts are not loaded eccentrically.* ACI 17.4.5.3

Factor for anchor bolts near an edge: $\Psi_{ed\_Na} := min\left(1.0, 0.7 + 0.3 \cdot \frac{c_{a1}}{c_{Na}}\right)$ $\Psi_{ed\_Na} = 0.918$ ACI Eqn. 17.4.5.4b

Factor for anchor bolts in un-cracked concrete near an edge without supplementary reinforcement: $\Psi_{cp\_Na} := min\left(1.0, \max\left(\frac{c_{a1}}{c_{ac}}, \frac{c_{Na}}{c_{ac}}\right)\right)$ $\Psi_{cp\_Na} = 0.733$ ACI Eqn. 17.4.5.5b

Wall caps are not under load, and per the wall cap design, service moment from post does not cause cracking.

Nominal bond strength of the adhesive anchor(s): $\phi N_{ag} := \phi_{adh} \cdot \frac{A_{Na}}{A_{Nao}} \cdot \Psi_{ec\_Na} \cdot \Psi_{ed\_Na} \cdot \Psi_{cp\_Na} \cdot N_{ba}$ $\phi N_{ag} = 7.192\ kip$ ACI Eqn. 17.4.5.1b

Check of bolt bond stress failure:

Bond_Stress_Check := if $\phi N_{ag} \ge n_{ab} \cdot T_{u\_ab}$
    "Bolt is satisfactory."
else
    "Bolt is no good."

Bond_Stress_Check = "Bolt is satisfactory."

Steel strength of anchor in shear (17.5.1)

Steel shear strength of anchor is confirmed above; so, no check here is necessary.

Concrete breakout strength of anchor in shear (17.5.2)

Check bolt group action for shear concrete breakout: ACI 17.2.1.1

Group_Shear_Breakout_Check := if $s_1 \le 3 \cdot c_{a1}$
    "Bolts act in group."
else
    "Bolts act singly."

Group_Shear_Breakout_Check = "Bolts act in group."

Theoretical projected influence area of a single bolt far from an edge: $A_{Vco} := 4.5 \cdot {c_{a1}}^2$ $A_{Vco} = 127.122\ in^2$ ACI Eqn. 17.5.2.1c

Actual projected influence area for bolt(s): $A_{Vc} := min\left(1.5 \cdot c_{a1} \cdot \left(1.5 \cdot c_{a1} + min\left(s_1, 3 \cdot c_{a1}\right) + min\left(1.5 \cdot c_{a1}, c_{a2}\right)\right), n_{ab} \cdot A_{Vco}\right)$ $A_{Vc} = 155.245\ in^2$ ACI Fig. R17.5.2.1b

Load bearing length: $l_e := h_{ef}$ $l_e = 5\ in$ ACI 17.5.2.2

Basic concrete breakout strength in shear for single anchor:

$$V_b := min\left(\left(7\left(\frac{l_e}{d_a}\right)^{0.2} \cdot \sqrt{\frac{d_a}{in}}\right) \cdot 1.0 \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{c_{a1}}{in}\right)^{1.5}, 9 \cdot 1.0 \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{c_{a1}}{in}\right)^{1.5}\right) \cdot lbf \quad V_b = 6.5\ kip$$

Concrete is not light weight; so, lambda-a is set to 1.0. ACI Eqns. 17.5.2.2a & 17.5.2.2b

Factor for eccentrically loaded anchor bolts: $\Psi_{ec\_V} := 1.0$ *Anchor bolts are not loaded eccentrically.* ACI 17.5.2.5

Factor for anchor bolts near an edge: $\Psi_{ed\_V} := min\left(1.0, 0.7 + 0.3 \cdot \frac{c_{a2}}{1.5 \cdot c_{a1}}\right)$ $\Psi_{ed\_V} = 0.869$ ACI Eqns. 17.5.2.6a & 17.5.2.6b

Factor for anchor bolts in un-cracked concrete: $\Psi_{c\_V} := 1.4$ *Wall caps are not under load, and per the wall cap design, service moment from post does not cause cracking.* ACI 17.5.2.7

Factor for small embedment $\Psi_{h\_V} := min\left(1.0, \sqrt{\frac{1.5 \cdot c_{a1}}{h_{ef}}}\right)$ $\Psi_{h\_V} = 1$ *ACI Eqn. 17.5.2.8*

Nominal concrete shear breakout strength: $\phi V_{cbg} := \phi_{adh} \cdot \frac{A_{Vc}}{A_{Vco}} \cdot \Psi_{ec\_V} \cdot \Psi_{ed\_V} \cdot \Psi_{c\_V} \cdot \Psi_{h\_V} \cdot V_b$ $\phi V_{cbg} = 6.28$ ***kip*** *ACI Eqn. 17.5.2.1b*

Check of concrete shear breakout failure:

$$Concrete\_Shear\_Breakout\_Check := \text{if } \phi V_{cbg} \geq V_{post\_u} \;\; \| \text{“Bolt is satisfactory.”} \;\; \text{else} \;\; \| \text{“Bolt is no good.”}$$

$Concrete\_Shear\_Breakout\_Check =$ “Bolt is satisfactory.”

Concrete pryout strength of anchor in shear (17.5.3)

Basic concrete pryout strength of a single anchor in shear: $\phi N_{cpg} := min\left(\phi N_{ag}, \phi N_{cbg}\right)$ $\phi N_{cpg} = 7.192$ ***kip*** *ACI 17.5.3.1*

Concrete pryout strength in shear coefficient: $k_{cp} := \text{if } h_{ef} < 2.5 \textit{ in} \;\; \| 1.0 \;\; \text{else} \;\; \| 2.0$ $k_{cp} = 2$ *ACI 17.5.3.1*

Nominal concrete pryout strength of anchor(s) in shear: $\phi V_{cpg} := k_{cp} \cdot \phi N_{cpg}$ $\phi V_{cpg} = 14.384$ ***kip*** *ACI Eqn. 17.5.3.1b*

Check of concrete pryout strength in shear:

$$Concrete\_Shear\_Pryout\_Check := \text{if } \phi V_{cpg} \geq V_{post\_u} \;\; \| \text{“Bolt is satisfactory.”} \;\; \text{else} \;\; \| \text{“Bolt is no good.”}$$

$Concrete\_Shear\_Pryout\_Check =$ “Bolt is satisfactory.”